

**Enhancing bridge construction and fiscal efficiency**

by

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The student author, whose presentation of the scholarship herein was approved by the program of study committee, is solely responsible for the content of this thesis. The Graduate College will ensure this thesis is globally accessible and will not permit alterations after a degree is conferred.

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**NOMENCLATURE**

AASHTO	American Association of State Highway and Transportation Officials
ABC	Accelerated Bridge Construction
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
BDM	Bridge Design Manual
CCS	Continuous Concrete Slab
CFT	Concrete Filled Tube
DOT	Department of Transportation
FHWA	Federal Highway Administration
I/D	Incentive/Disincentive
NCSC	North Central States Consortium
NHS	National Highway System
OBS	Office of Bridges and Structures
OC	Office of Contracts
PBES	Prefabricated Bridge Elements and Systems
PPCB	Pretensioned Prestressed Concrete Beam
RCE	Resident Construction Engineer
ROW	Right of Way
SHA	State Highway Agency
TRB	Transportation Research Board
VE	Value Engineer

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**ABSTRACT**

With the growing demands facing public infrastructure and the shortage of necessary funds, State Highway Agencies (SHA) across the nation are increasingly being forced to explore cost-effective, safe, and efficient solutions to implement in response to these challenges. The high cost, safety factors, and out-of-service and user cost considerations make bridges and their associated work a particular focus of many in the industry. This thesis highlights two areas within the field of bridge engineering design and management that will continue to aid SHAs in their goal of producing safe, efficient, and innovative products that serve the public well.

Substructure bridge components are designed to resist gravitational forces such as dead load and vehicular live load, as well as lateral forces including wind, vehicular braking and centrifugal force effects. Significant lateral forces can create “uplift” conditions on some portions of the foundation. A review of current design techniques regarding uplift in the oft used pile-to-pile cap connection indicates a lack of uniformity in the design process across state agencies stemming from minimal research performed in this area. Additionally, approved uplift anchors for use in the field have not been tested. In order to close this gap, twenty-one full scale steel H-pile specimens were fabricated and tested in Iowa State University’s Structural Engineering Laboratory to test the capacity of the pile-to-pile cap connection under static tensile loading. Findings revealed that 1) capacity of bare piles is generally underestimated and could be more frequently considered for uplift design; 2) concrete cracking leads to a loss of bond in these types of connections; and 3) positive anchorage and/or embedment that extends above the lower rebar mat of the footing is necessary to develop a high capacity connection.



In conjunction with the proposed safety improvements in design practice for pile-to-pile cap connections subject to uplift, the impact of bridge projects, specifically on the traveling public in terms of user costs, is an area that bears further consideration and scrutiny. Barring the use of Accelerated Bridge Construction (ABC), bridge projects are generally let with long contract durations that can be on the order of three months to several years. The goal of this methodology is to accommodate traditional construction practices and produce a competitive, low cost project. The fact remains, however, that the presence and duration of these projects often poses significant obstacles and interruptions to the public. The possibility of accelerating these projects while maintaining traditional construction methods is thus explored, the likes of which have been minimally investigated. A review of historical data from the Iowa Department of Transportation (DOT) along with interviews with both DOT personnel and contractors were carried out to help shed light on the feasibility and cost associated with this acceleration activity. Findings revealed that traditional construction methods can accommodate significant degrees of acceleration (up to 50-75%). The cost associated with this compression activity is highly variable though and may be driven more by external factors (inspector quality, risk, contractor ability) than specific, project-based constraints.

## **CHAPTER 1. INTRODUCTION**

The last one hundred years has seen incredible advancements in the field of transportation in the United States of America. More specifically, bridge engineering and construction practice have changed dramatically. From the steel truss systems popular in the early 1900s to the wide implementation of pre-stressed concrete bridges after WWII and more recently the introduction of Accelerated Bridge Construction (ABC), technologies and techniques have always evolved to meet the present demand of the times. Today that need has perhaps never been clearer. With budgets on the decline along with facing an infrastructure that has been aging since the 1960s and 1970s, State Highway Agencies (SHA) are increasingly looking for solutions to cut down on costs and improve efficiency, while still maintaining the production of safe and reliable structures.

The scope of infrastructure problem is vast and cannot be understated, and as such it should be recognized that positive solutions will most often address small niches of this overarching issue. Advancements in efficiency and innovation – in terms of dollars, design, and schedule – through research and monitoring the current and past design/construction practice have been made. This thesis seeks to build on the work of past researchers while proposing solutions to two specific areas of bridge engineering and management, which will improve both the safety of the structures, as well as maximize cost/schedule aspects of bridge projects.

### **Uplift Considerations in the Pile-to-Pile Cap Connections**

One of the most common configurations for bridge pier foundations is the use of driven piles with concrete cap (i.e. pier footing) cast on top. This type of construction results in a cost effective solution to transfer gravity and lateral loads acting on the superstructure into the substructure components and the surrounding soil. Little attention has been previously given to

the performance of the pile-to-pile cap connection subject to uplift forces, which are generated by lateral loads such as wind, seismic, and temperature, as well as unbalanced gravity loading. Perhaps due to the non-intuitive existence of these “pull-out” type forces, few agencies have developed tested and proven design rationale. The most common solution to account for uplift forces in the connection is to secure some type of transverse bar reinforcement to the web of the pile, usually in the form of welded studs, rebar, or threaded rods. Some agency’s standard practice is to ignore the uplift condition altogether (i.e. dead weight is increased or designs are modified so that any uplift forces are negated). Opting for the latter approach can prove expensive and is sometimes impractical given local site and project conditions.

With the advent of larger structures that utilize tall piers and long spans, uplift loads are becoming more prevalent in design, highlighting the need for concrete, evidence based methods of design practice. Chapter 2 will address this concern, and provide several cost effective solutions that designers and contractors can implement to enable the safe transfer of loads in pile-to-pile cap connections with uplift considerations.

### **Acceleration in Traditional Bridge Projects**

Much attention has been given to the practice of ABC construction in the past 10-20 years in the United States, and for good reason. With rising traffic volumes and increased user costs, ABC has provided a way to do rapid rehabilitation and construction on bridges with minimal impact to the traveling public. However, implementation of these techniques cannot, at least at this point in time, be performed for all or even most projects in an agency’s network; the cost and resources required are simply too great. The demand for reduced road closure periods and accelerated projects remains high despite this. One possible solution to serve a broader range of an agency’s assets is to manipulate (reduce) contract/road closure durations for bridge projects

utilizing traditional construction techniques. The cost associated with this compression activity is still unknown and the extent of previous investigation into this field is minimal.

Most state agencies let projects and award bids based on a 'lowest-bid' metric, as they are public entities that are funded through taxpayer dollars. In an effort to keep bid prices low, agencies will estimate the appropriate time to complete the work contained in the contract, or air on the flexible side so as to give contractors plenty of time to schedule their crews and equipment efficiently. The emphasis on the cost aspect may therefore be to the detriment of any schedule savings. In order to help agencies better understand the relationship between cost and time as acceleration is introduced on bridge projects, Chapter 3 includes an analysis of historical data along with contractor interviews. This provides a framework from which agencies can begin to properly evaluate all project constraints and best choose where to allocate their resources the most efficiently.

## **CHAPTER 2. PERFORMANCE INVESTIGATION AND DESIGN OF PILE-TO-PILE CAP CONNECTIONS SUBJECT TO UPLIFT**

### **Introduction**

While not desirable, it is common for some amount of substructure uplift force to be generated under certain loading conditions. In Iowa, uplift conditions are usually produced from the Strength 1, Strength 3, Strength 5, Extreme Event 2, and Service 1 load combinations found in the American Association of State and Highway Transportation Officials (AASHTO) bridge design code (AASHTO 2014). Contributing factors include braking, centrifugal force effects, stream forces, temperature, wind, and unbalanced vertical loading. In general, continuous multi-span bridges that incorporate tall piers and long spans have larger uplift forces. Tall piers increase the overturning effect of lateral loads as a result of a larger moment arm. In addition to this, fixed piers on long continuous multi-span bridges typically must resist proportionally more longitudinal loading than expansion piers, resulting in a higher probability of uplift forces. Curved structures can also introduce thermal and friction forces acting transversely, further increasing potential uplift loads transferred to the substructure elements (A. Nelson, personal communication, July 2017).

An investigation of recent Iowa DOT bridge designs show that factored uplift values used in design are in the range of 8-40 kips for one pile. A select few projects on interstate ramps have been designed for uplift values as high as 87 kips (Iowa DOT). In the event of such uplift forces the Iowa DOT has a clear design methodology for handling pile to soil interaction (OBS 2017). However, once the pile design has been satisfied, attention must be focused on the critical pile-to-pile cap (or pier footing) connection. Very little literature is available on this topic, which has led to a broad range of design approaches that are in effect today. With the larger uplift loads

now found in design, it is imperative that details used to resist these forces be based upon sound experimental and analytical research coupled with engineering fundamentals.

### Current Practice

In Iowa the overwhelmingly preferred foundation configuration for piers and abutments consists of driven steel piles with a concrete cap (OBS 2017). Figures 2.1 and 2.2 detail a pile foundation which is a representative example of current Iowa projects. Piles are positioned in a rectangular pattern around the base of the column and embedded 12" into the footing. In the case of pier piles, the perimeter piles are usually battered at an angle of 1 horizontal to 4 vertical to resist lateral forces acting on the superstructure. A lower mat of reinforcing steel is placed 1" clear above the tops of the piles and is generally composed of reinforcing bar sizes ranging from #8s to #11s. A top mat typically consisting of #5 reinforcing bars is included when uplift forces are present (OBS 2017).

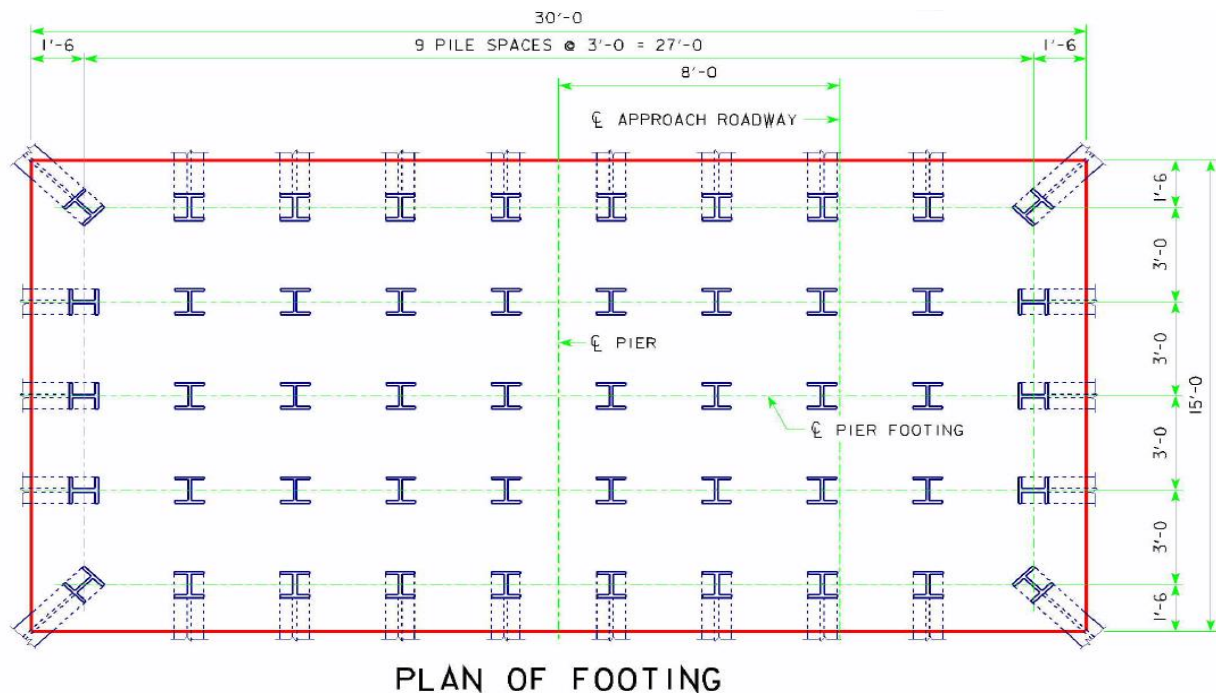


Figure 2.1 Typical bridge and substructure details: Plan view

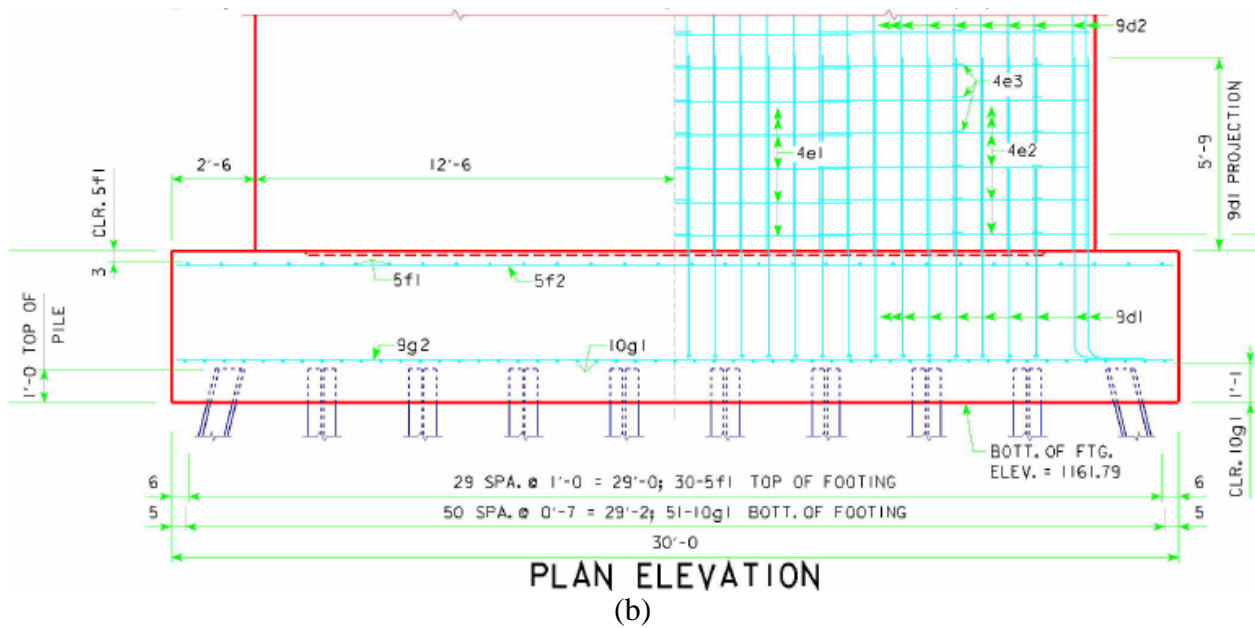


Figure 2.2 Typical bridge and substructure details: Elevation view

Corner piles are often susceptible to the highest uplift forces. The Iowa DOT currently assumes a nominal bond strength of 0.060 ksi between the pile and pile cap concrete if uplift forces are expected. This nominal bond strength is based on the bond stress recommended in the American Institute of Steel Construction (AISC) design manual for filled steel columns and is thought to be a reasonable lower bound as mentioned in the design manual commentary (OBS 2017, AISC 2011). Two other papers pertaining to bond stress, one by Griffis (1992) for encased steel columns and another by Watson (1974) related to embedded beam flanges, recommend even higher values.

For design, the resistance factor for the strength limit state check is set at 0.45 per the recommendation in AISC (2011) for bond stress on filled steel columns. While the AASHTO code generally specifies a resistance factor of 1.0 for Extreme Event, a 0.75 factor used by the Iowa DOT acknowledges the critical nature of the connection (AASHTO 2014, GAI 1982, Pertold et al. 2000).

In the event that the bond strength is not sufficient, positive anchorage must be attached to the pile. The Iowa DOT specifies the use of an F1554 threaded rod through the web of the pile or the use of welded studs to the web. However, a brief survey of recent bridge plans in Iowa showed a range of anchors utilized including, but not limited to, C-channels, L angles, and in one instance V-bars through the web (Iowa DOT).

Results from the 2016 North Central States Consortium (NCSC) questionnaire, in which ten state agencies answered questions relating to current design practice for bridges, revealed that methods for uplift design in the pile connection varied greatly (G. Novey, personal communication, October 2016). Pertaining to bare piles subject to uplift forces, Iowa's specification of 0.060 ksi is the largest allowed of all NCSC respondents. Other states either do not consider any uplift capacity, or assume some lesser bond strength (i.e. 1.0 ksf or 10 psi). Relating to anchorage, welded studs to the web of the pile or drilling/torthing holes and inserting a bar perpendicular to the pile are the most commonly used. The methodology for setting the ultimate capacity for these anchors is either unknown and assumed or based on the breakout capacity for studs in shear found in ACI 318. North Dakota welds reinforcing steel directly to the pile (longitudinal orientation) using straight bars and sets the yield strength for the bars as the capacity of the connection. The remaining few states do not have uplift details and instead prefer to resize the footing so that no uplift is generated. While resizing the footing is a viable option, designing piles for no uplift is sometimes impractical/impossible and may not be cost efficient. Therefore, appropriate positive anchorage details must be incorporated into design manuals for use, even if the allowable uplift forces are small.

### **Previous Investigation**

Two notable studies have been completed to experimentally evaluate pier piles subjected to uplift forces. In 1972 Horn, Stewart, and Boulware tested six different anchorage details



attached to steel and timber piles (Horn et al. 1972). For each detail three specimens were fabricated and tested, twice under static loading and once under 100 cycles of loading equal to 40% of the bearing capacity of the pile. The steel pile anchors consisted of hooked plates bolted to the web and #6 V-bars through the web. They attained average ultimate static capacities of 76-125 kips with 5" of embedment. Current AASHTO code specifies that piles must 'project at least 12" into the pile cap after all damaged material is removed'. This standard is reduced to 6" if there is positive anchorage present (AASHTO 2014). Horn et al. also tested one bare steel pile which reached an ultimate capacity of 60 kips. Embedment depth for the control specimen is not clearly stated, but is assumed to be 6" or less based on other specimens tested. The research concluded that V-bars on steel piles were the most cost efficient and best performing anchorage device between the two that they tested.

GAI Consultants, Inc. (1982) recognized the need for further testing with their 1982 publication "The Steel Pile-to-Pile Cap Connection." Twelve HP10 and four PP10- $\frac{3}{4}$  x 0.5" piles with various anchorage details were subjected to static tensile forces to measure the capacity of the connection. Embedment depth ranged from 6-9" in unreinforced blocks and the testing plan included two bare piles. The most proficient performing anchors were straight bars that were welded to the web. The majority of the other anchorage types (cap plates, angles, and bar grids) performed marginally better or worse than the bare pile that was used as a control. They hypothesized that this was due to the fact that these types of anchors only applied load over a limited region near the tip of the pile, thus resulting in lower ultimate capacities. GAI concluded that 'the ultimate tension strength of a pile cap-pile anchor system is directly related to the depth of embedment of the tension anchor', rather than the shape of pile. As such, they recommend

welded straight or hooked bars be used over anchorage types that do not take advantage of reinforcing bar development length.

Several studies have been conducted on pile-to-pile cap behavior under lateral or simulated seismic loading (Rollins et al. 2010, Xiao et al. 2006, Shama et al. 2001). These studies largely focus on the moment capacity and load-displacement response with different details in the connection as well as retrofit analysis. Shama et al. (2000a, 2000b) investigated the behavior of the pile-to-pier cap under seismic loading as well. Pertold (2000a, 2000b) recognizes that emphasis has been placed on sections subject to bending moment and highlights the need for more information on the vertical resistance of embedded columns. His work isolates the effects of bond and punching resistances in footings. Significant resistance from bond stress is observed and is more than the punching resistance for the given geometry. However, no testing was completed to study the interaction of these two failure mechanisms acting simultaneously.

Heristchian et al. (2014a) compared the experimental and numerical capacities of three embedded IPE140 sections with and without baseplates. In conjunction with prior research, Heristchian (2014b) observes that the introduction of the end plate or utilizing tapered sections mobilizes a bearing force that has higher unit stresses than shear and is closely related to the punching resistance of the concrete. He concludes that more experimental data and numerical models on pullout behavior are necessary to verify and validate the results.

Preliminary work has been completed on pile-to-pile cap connections under uplift forces. Despite this, questions still remain about the ultimate capacities of specific connections used by various states and the current methodology for designing bare piles subject to uplift. Furthermore, from the NCSC report it is clear that design practice for uplift in the connection is not uniform or very well substantiated. Therefore, the goal of this research is to investigate an

often overlooked portion of substructure design and bring clarity to the process. Design values for current and higher capacity anchors are recommended, as well as design protocol for bare piles embedded in pile caps and subject to uplift forces.

### **Experimental Setup and Methods**

The focus of this work is on the ultimate capacity of pile-to-pile cap connections and, as such, emulating field conditions was critical. The testing layout and specimen dimensions utilized emphasize this while also seeking to minimize the effect of arbitrary stress introduction at the pile face, which is an important consideration (Heristchian 2014a). The test setup resembles that of Horn 1972, but includes a few modifications/improvements. The pile is cast horizontally between two simulated concrete footings measuring 4' x 4' x 3' and set on rollers. The lower reinforcing bar mat consists of #9's both ways spaced at 8.5" and 10" o.c. and the top mat is composed of #5's both ways spaced at 12" o.c. Dimensions and reinforcement details were taken from a sampling of representative Iowa projects and conform to the minimum standards given in the Iowa Bridge Design Manual and the AASHTO Bridge Design Manual. Figure 2 shows the layout schematic and detailed dimensions.

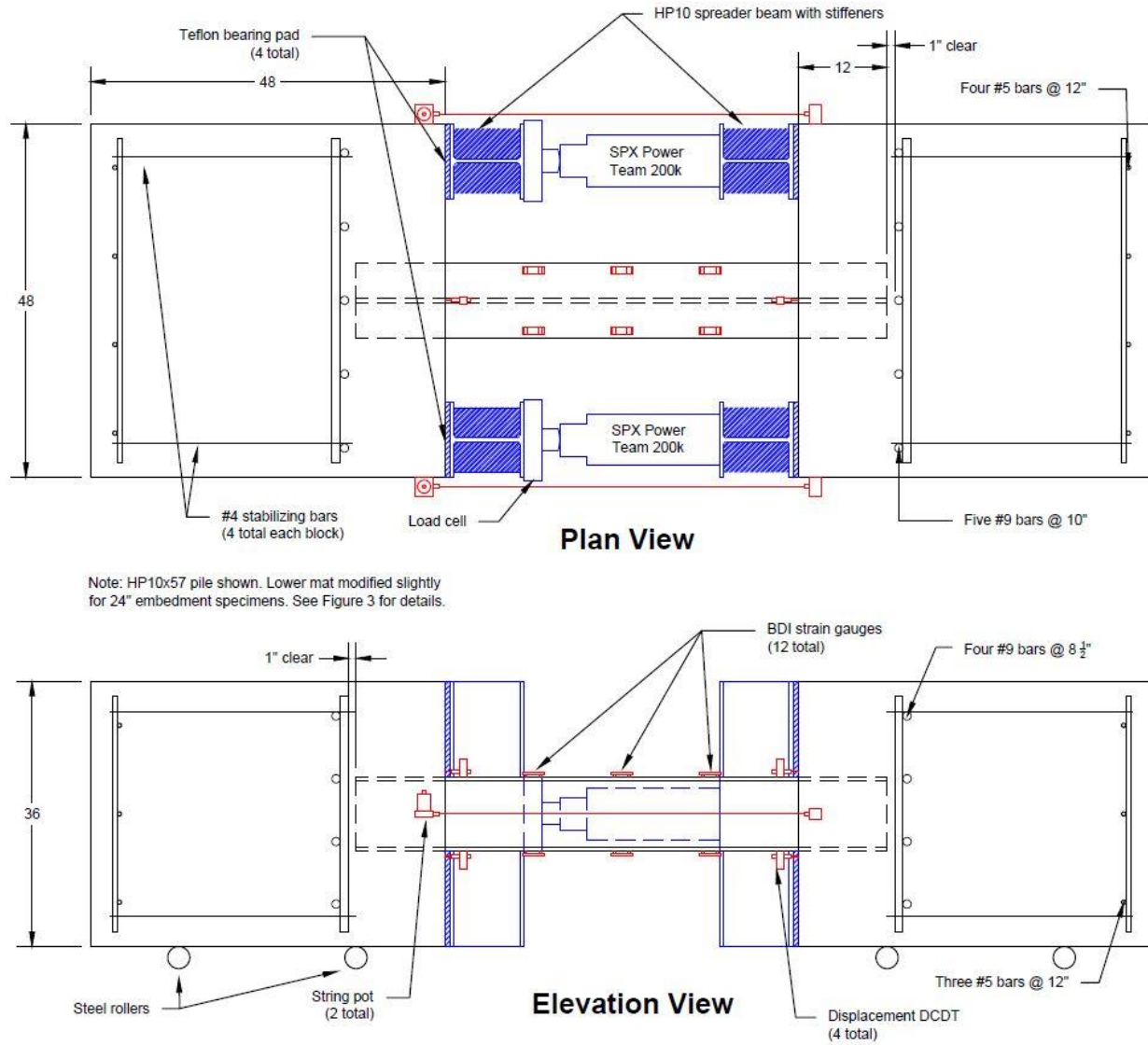


Figure 2.3 *Testing layout*

A major benefit of the test configuration is that each test produces the lower of two ultimate capacities for the connection being tested (one on each end of the pile) - essentially doubling the data pool. Two 200 kip actuator jacks are set at mid-height between the simulated footings to induce tensile forces on the pile. HP10x57 piles with stiffeners distribute the force as a line load on each side of the pile, resulting in lower unit stresses than the concentrated load forces applied by Horn (1972) and GAI (1982).

For all test specimens, a standard Iowa 4C 4,000psi mix design was used. Twelve 4"x 8" cylinders and three modulus beams were cast for each mix to obtain basic strength properties (results may be found in Table 2.1). Specimens were cured for at least seven days and obtained a minimum 4,000psi strength before moving to prevent cracking at the connection (with the exception of E5 and E6, which did not achieve design strength after 28 days).

To understand and quantify the type of loads being induced on the connections, twelve strain gauges were attached to the top and bottom flanges of the pile. From the recorded longitudinal strains, a system of developed equations was used to determine the axial, bending, and torsion stress induced on the pile at each of the three locations. In addition to the strain gauges, displacement was measured at the interface between the pile and footing on the top and bottom flanges with four displacement transducers. Values were averaged for each side and used for subsequent analysis. Two string potentiometers were also utilized to measure the relative movement between pile ends.

### **Anchorage and Pile Selection**

Three different size piles were utilized in this work (HP14x73, HP10x57, and HP10x42) to investigate the effect of size and surface area on the ultimate capacities. Piles were obtained from existing bridges to mimic worst-case material conditions, but were visually inspected to ensure straightness and no discontinuities in the embedded portion of the connection. The surfaces of the embedded portions were dry-broomed but were otherwise left unprepared.

Fabrication and testing were completed in groups of two or three specimens, which allowed an iterative approach to be taken in developing the anchorage details. In total, seven different connections were tested with varying degrees of replication. Each connection is discussed briefly below with the correlating detailed sketches given in Figures 2.4-2.7. Design of

the anchors was based on published literature, input from Iowa DOT designers, and the research team. Several factors were considered, including effectiveness of connection, ease of installation, no field welds (due to the high variability in quality), and expected ultimate failure. Six piles were embedded 12" with no additional anchorage in order to serve as control specimens and to evaluate the current bare pile design methodology. The current approved anchor in Iowa was tested, as well as several piles confined with hoops to increase crack control. Embedment was investigated by increasing the depth to 24" as this is a relatively easy alternative to fabricating positive anchorage. In addition, more involved anchors consisting of U and V-bars tied to the top mat provided insights into the performance of deeply embedded anchors. In all tests of details that had torched or drilled holes, wood or plastic spacers were installed between the bar and the top of the hole to simulate poor construction quality.

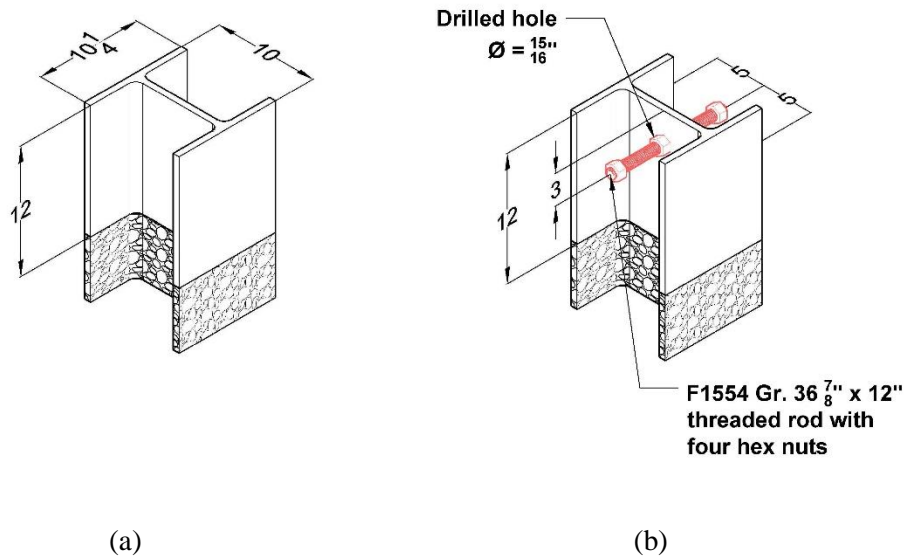
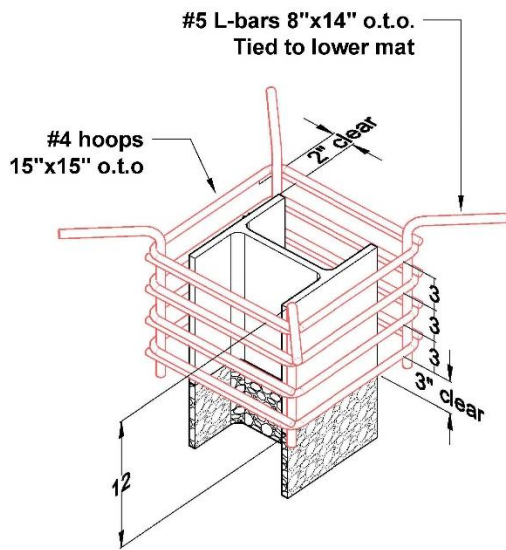
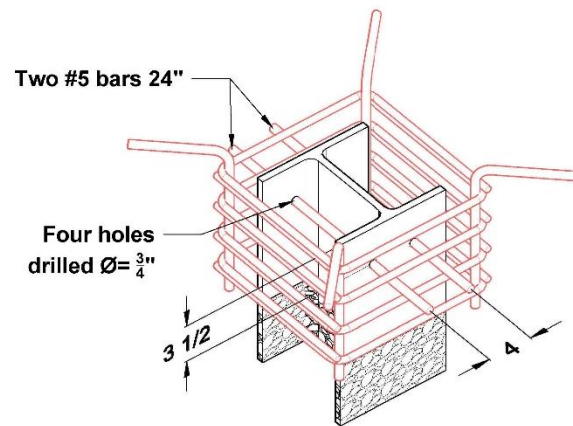


Figure 2.4 Sketch of anchor details: (a) No Anchor, (b) Threaded Rod



(a)



\*Dimensions same as cage detail, with additional straight bars as shown

(b)

Figure 2.5 Sketch of anchor details: (a) Cage, (b) Cage with Bars

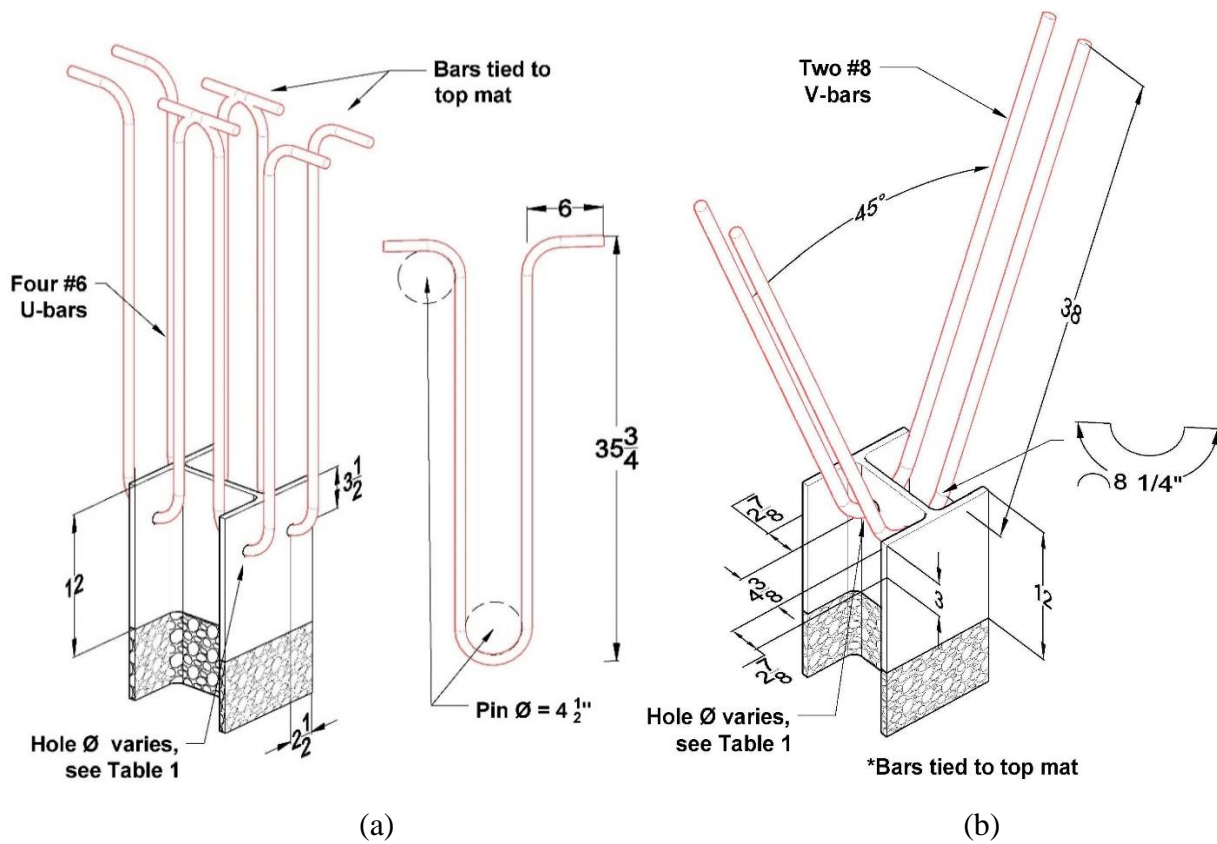


Figure 2.6 Sketch of anchor details: (a) U-Bar, (b) V-Bar

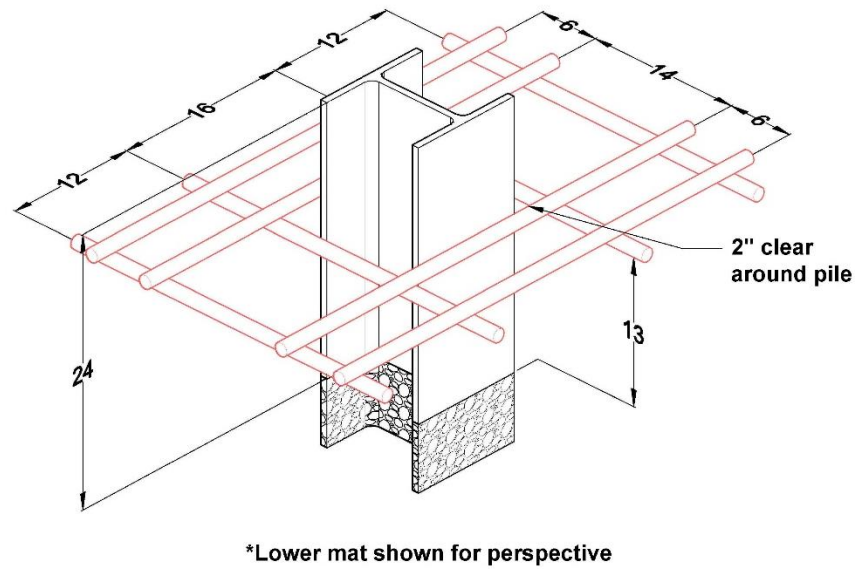


Figure 2.7 Sketch of anchor details: 24" Embedment

### Test Results and Discussion

Results from the testing are summarized in Table 2.1. For completeness, concrete strength properties are included for each specimen, but were not the primary focus of this research. For reference, the relationship between bond and concrete compressive strength for plain bars was considered by Feldman (2005). Slip data from both ends of the specimen were measured and analyzed; however only data from the block that failed are included in Table 2.1 and subsequent figures. Due to the type of cracking observed on some of the specimens, recorded displacements after the ultimate load was attained may not be fully representative of the overall behavior (these specimens are marked with a footnote in Table 2.1).



Table 2.1 *Test Results*

Specimen ID	Pile Size	Anchor Type	$f'_c$ (psi)	$f'_r$ (psi)	Ultimate Load (kips)	Slip @ Ultimate Load (in)
NA1	HP14x73	No Anchor	5900	570	107	0.0171
NA2	HP14x73	No Anchor	4900	490	114	0.011
NA3	HP14x73	No Anchor	4700	510	123	0.0376
NA4	HP10x57	No Anchor	6100	560	127	0.0122
NA5	HP10x57	No Anchor	4500	490	125	0.0124
NA6	HP10x57	No Anchor	4500	490	108	0.0145
TR1	HP14x73	Threaded Rod	5900	570	104	0.0131
TR2	HP14x73	Threaded Rod	4900	490	112	0.0121
C1	HP10x57	Cage	5900	500	136	0.0206
C2	HP10x57	Cage with Bars	5900	500	145	0.0105
E1	HP10x42	24" Embedment	5700	560	218	0.0534
E2	HP10x42	24" Embedment	6200	610	175	0.017
E3 <sup>a</sup>	HP10x57	24" Embedment	4600	540	208	0.0146
E4 <sup>a</sup>	HP14x73	24" Embedment	4300	500	190	0.0090
E5 <sup>a</sup>	HP14x73	24" Embedment	3600	430	177	0.0088
E6 <sup>a</sup>	HP14x73	24" Embedment	3600	430	182	0.0052
U1	HP10x42	U-Bar (1" Drilled Holes)	5800	560	277	0.0507
U2	HP10x57	U-Bar (1-1 ¼" Torched)	7000	680	257	0.0686
V1	HP10x57	V Bar (1 ¼ - 1 ½" Torched)	7000	680	236	0.0385
V2 <sup>a</sup>	HP10x57	V Bar (2 ½" Torched)	4900	570	203	0.0158
V3 <sup>a</sup>	HP10x57	V Bar (2" Torched)	4900	570	198	0.0128

(a) Displacement after ultimate load may be suspect

## Bare Pile Tests

All six tests with piles embedded 12" into the footing attained substantially higher ultimate capacities than the current design methodology that assumes a 60 psi bond stress (see Figure 2.8). Size of the pile did not seem to affect the behavior of the connection nor the ultimate capacity for the HP10 and HP14 piles considered here. Initial stiffness values varied and ranged from 50,000-95,000 k/in, and all specimens behaved linearly until 60-70 kips. At that load all specimens had a notable stiffness change. Within each test, displacement between blocks for the most part tracked similarly and then started to deviate between 65 and 90 kips. The overall magnitude of slip at the ultimate load was small and similar for both pile sizes.

The failure was characterized by a sudden formation of diagonal cracks propagating from the pile while load simultaneously dropped to between 0 and 12 kips. Specimen NA3 was able to sustain slightly larger displacements than the other specimens near their ultimate loads, but still displayed a brittle failure. The observed behavior is consistent with previous work (Pertold 2000, Horn 1972). A typical failure mode for bare piles is illustrated in Figure 2.9.

Further investigation revealed that concrete cracking leads to a loss of bond between the pile and concrete, suggesting that design methods based on bond stress may not be accurate. In other words, the concrete is not sufficiently confined and/or the pile embedded deep enough so as to allow for a pure bond/pullout failure. Fundamental evidence of this fact are the results which show HP14 piles reaching ultimate capacities at or below a HP10 for the same embedment depth, even though HP14s have a 40% larger surface area.

From the testing of the 12" embedded specimens it became apparent that the limiting factor in the connection rests upon crack initiation in the concrete, which in general is impacted by the strength of the concrete, footing geometry, anchorage (if applicable), and the shape and size of section. With respect to strength, Roeder concludes in his study of composite action in circular and square concrete-filled tube (CFT) sections that "bond stress capacity does not appear to be related to concrete strength" (Roeder 1999). Based on the apparent lack of a relationship between concrete strength and connection capacity, a similar conclusion might be drawn about the ultimate capacity. For example, specimens NA5 and NA6 were cast from the same ready-mixed concrete truck and have the same strength properties, yet the capacity differs by almost 20 kips. However, because the failure modes more closely resemble punching failure than bond failure, a strong correlation between concrete strength and ultimate capacity would be expected, as Hallgren (1998) notes. The absence of a complementary relationship prompted consideration

of the impact of other factors on the connection. Current design equations related to bond, like those put forth by Griffis (1992), only account for the influence of concrete strength. Likewise, punching shear failure design relates to strength, embedment depth, and the critical shear perimeter. A conservative and safe nominal capacity can be drawn from the data presented, but more investigation and research is needed. It is hypothesized that regardless of design protocol, this type of connection carries with it a higher degree of variability which should be reflected during design.

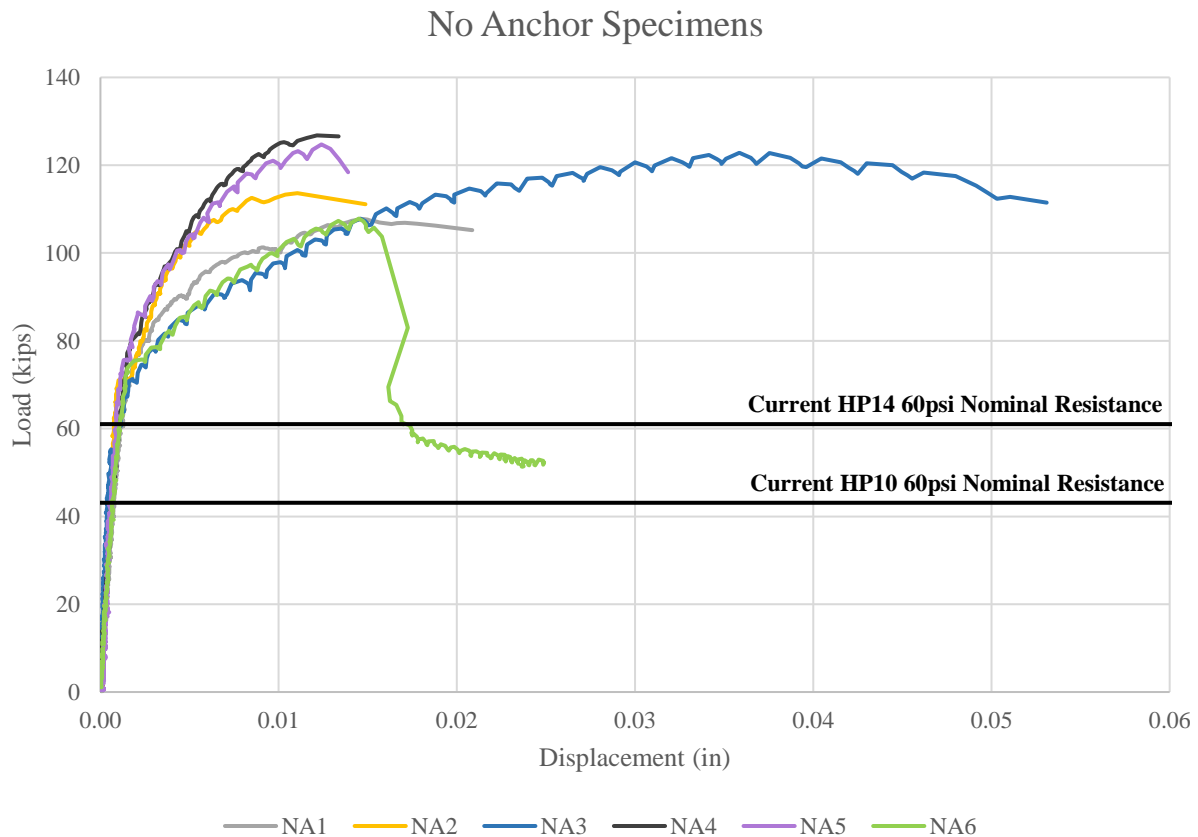


Figure 2.8 Test results: No Anchor control specimens

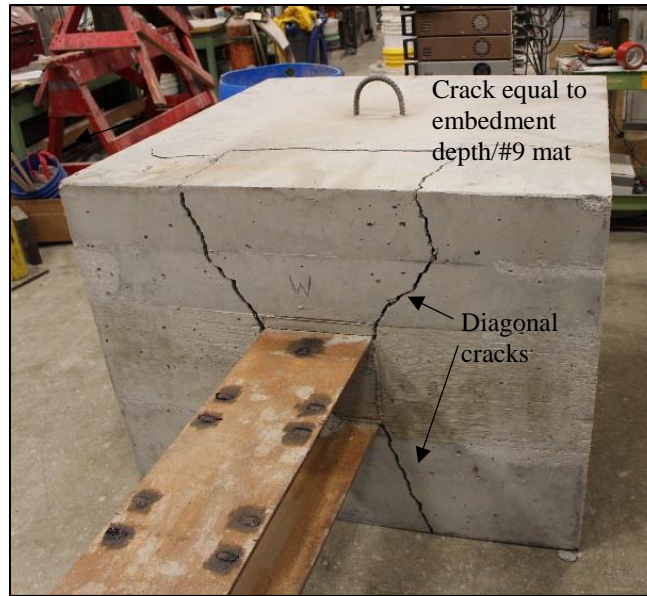


Figure 2.9 *Typical failure mode for No Anchor and Threaded Rod specimens*

### **Threaded Rod and Cage Specimens**

Load vs. Displacement curves for the threaded rod and cage specimens are presented in Figure 2.10. The threaded rod specimens (TR1 and TR2) behaved similar to the bare pile tests and achieved marginally lower ultimate capacities than their bare pile counterparts (NA1 and NA2). The cage and cage with bar specimens gave some indication of what impact confining the pile has on connection capacity. No noticeable difference was observed until after the peak load was reached for the cage specimen, highlighted in the lower right hand portion of Figure 2.10. The hoops performed adequately, confining the diagonal cracks and allowing the pile to slip out in a more bond-failure type fashion. At  $\frac{1}{2}$ " displacement the connection still maintained a load of 50 kips. The pile slipped out completely just before 0.6" of slip was reached at about 45 kips. With the additional bars through the flanges, specimen C2 obtained a slightly higher capacity but failed very suddenly. Concrete around the pile was confined, but a large shear plane along the hoops extending to the lower mat caused failure in the connection.

While confining the concrete increased the capacity of the connection to some extent and provided helpful insights, the capacity gain was small for the complexity of the design details. In addition, all specimens including the bare piles failed to engage the lower mat steel, as evidenced by cracking at the depth of the mat/pile. With the exception of specimen C1, anchorage only succeeded in pulling out a larger area of concrete around the pile, and consequently more closely resembles punching failure of a footing. These data are consistent with other anchors tested by GAI Consultants (cap plates, angles, and bar grids) and further supports the hypothesis that anchorage whose primary resisting mechanism is bearing near the tip of the pile is not effective and may actually be detrimental to the connection (GAI 1082, Heristchian 2014a). Pertold's findings (2000a) agree with this as he states that the "punching resistance for a certain geometry of the concrete base could be less efficient at resisting the vertical load from column than bond resistance." Based on these findings, it appears that anchorage design should prioritize embedment depth and consider crossing the lower mat in order to engage the steel and the rest of the footing.

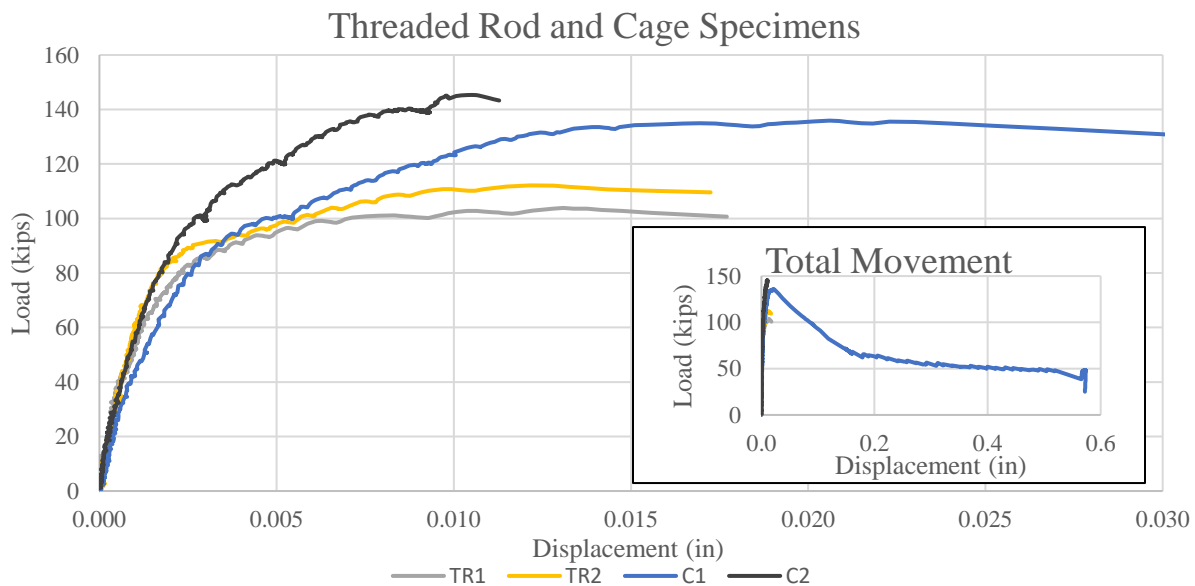
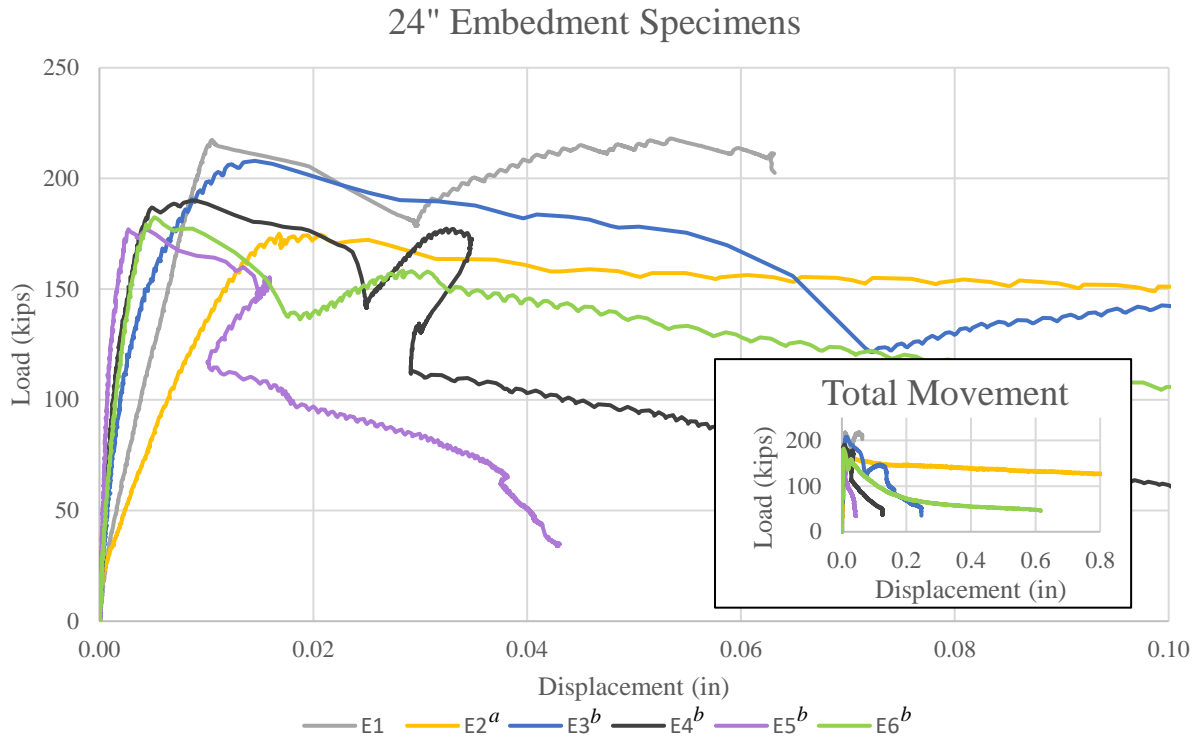


Figure 2.10 Test results: Threaded Rod and Cage/Cage with Bar

### **Connections with Deeper Embedment Details**

Results from the specimens incorporating deeper embedment performed significantly better than the previously discussed specimens, as shown in Figures 2.11 and 2.12. The 24” embedded specimens demonstrated a wide range of ultimate capacities bookended by two of the three HP10 tests. Specimen E1 had the highest capacity of 218 kips as shown in Table 2.1. Specimen E2 exhibited behavior most closely associated with pure bond pullout failure, with minimal cracking and very large displacements. At termination of the test, the load was 125 kips with a measured slip of 5” (instrumentation was only accurate to displacements equal to 1 3/8” in magnitude; final measurements recorded by tape), as shown in 2.11 and 2.13a. The ability of the connection to sustain significant load after initial movement can most likely be attributed to the confinement provided by the lower mat of #9 bars. Characteristic of the three tests on the HP14 piles was a tension crack that formed at the center of the face after the peak load was attained which caused the concrete to spall outward away from the pile (Figure 2.13b).

Concrete strength and stiffness of the connection did not correlate with the measured ultimate capacities that resulted. All 24” embedment specimens failed in a ductile manner except one (E1). Performance seems to rest largely on concrete strength and the ability of the lower mat to confine the pile. Additionally, consideration of punching failure through the top of the footing due to compression should be accounted for in piles that extend above the lower mat. Pestotnik (1979) recommends at least 1’ 6” of clear distance between the tip of the pile and the top of the footing. Hallgren’s research (1998) demonstrates that footings without vertical shear reinforcement still have considerable resistance to punching failure.



(a) Measured displacement at end of test = 5" (b) Displacement after ultimate load larger than indicated by curve

Figure 2.11 Test results for specimens with deeper embedment: 24" Embedment

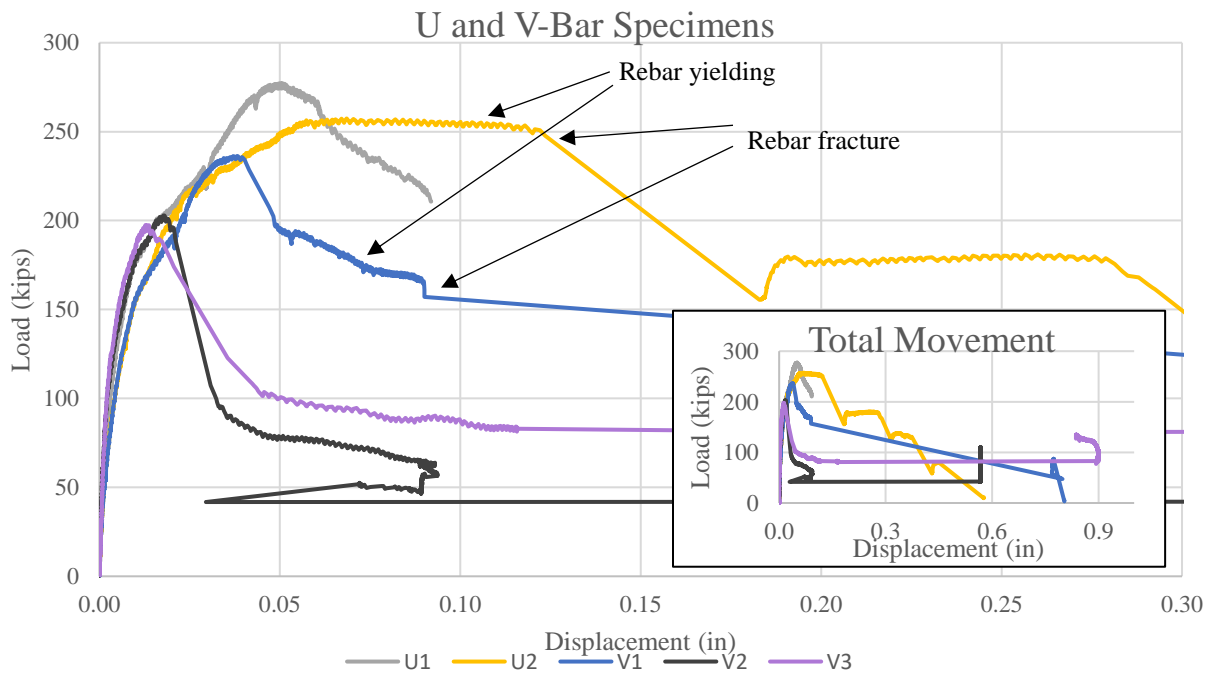


Figure 2.12 Test results for specimens with deeper embedment: U-Bar and V-Bar

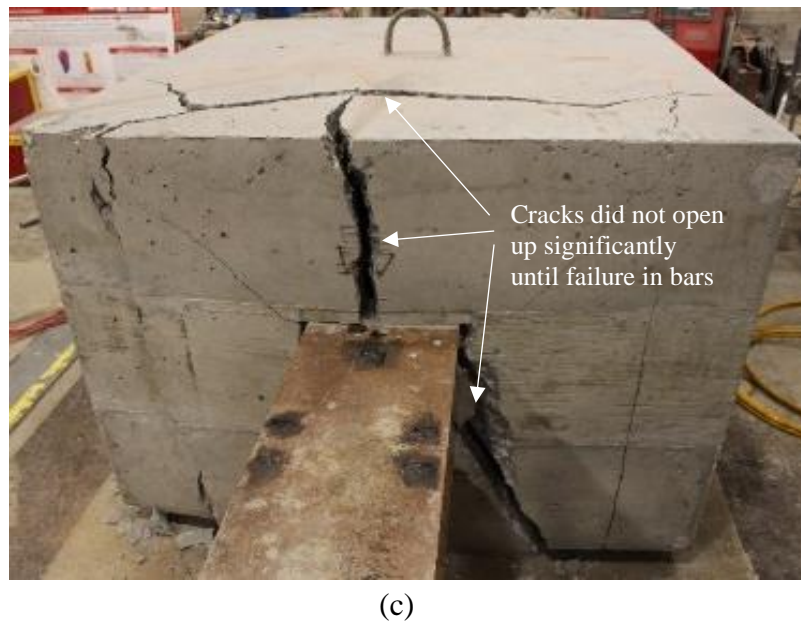
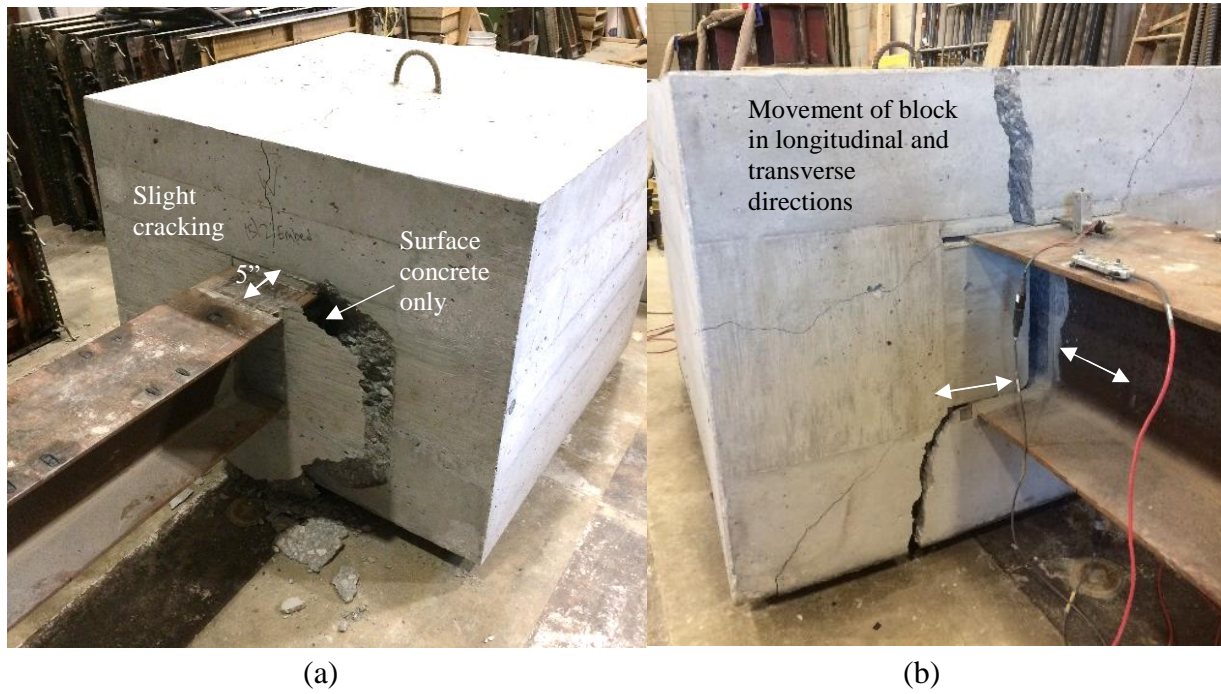


Figure 2.13 Failure modes: (a) Bond failure in specimen E2, (b) Tension crack typical in HP14 specimens with 24" embedment, (c) U1 and U2 typical cracking

Anchorage employing V and U-Bars extending to the top reinforcing steel mat produced the highest strength connections (Figure 2.12). In Specimen U1 initial diagonal cracking was



observed at 180 kips. The connection then resisted a peak load of 277 kips, well exceeding the theoretical yield strength of the bars, while demonstrating resistance against failure due to concrete cracking (2.13c). Strain hardening was observed in the bars and subsequent failure of the connection occurred when all four bars fractured simultaneously at the flange-hole interface.

Observations can be made concerning hole construction method, size, and placement in the web from specimens U2 and V1-3. U2 holes were torched as specified previously. The overall capacity was about 20 kips less than U1, but as Figure 2.12 shows the steel did not engage uniformly, and as a result bars fractured at four distinct points. Torched holes were incorporated in all V-bar specimens because of constructability concerns with drilling in the web of smaller piles. Specimen V1 bars also did not engage uniformly due to torching, but still attained substantial peak load and failed due to reinforcing steel shear at the hole interface. V2 and V3 were fabricated with “large” torched holes. No failure of the rebar was observed in contrast to the other specimens, although the large jump in displacement near the end of the test could indicate a fracture of one of the bars. After this point the load began to increase, suggesting at least one bar remained attached to the pile to provide resistance. Testing was terminated shortly after because the concrete had sustained excessive cracking which compromised the test setup. All three V-Bar specimens still surpassed the yield strength of the bars. As long as there is sufficient bond and development length is ensured, testing demonstrated that these types of connections are perhaps the most efficient and reliable. If hole size is controlled, the failure mechanism is transferred from concrete cracking to steel bar fracture, which may be preferable.

Concerning design strength, even though bars in these types of connections failed in shear at the hole interface rather than in tension in the legs, a good lower bound metric for estimating the capacity of the connection seems to be the total area steel embedded (i.e. number

of bar legs times area of one bar times bar yield strength). More experimental and numerical studies with different bar size and quantity included are necessary in order to develop models to more accurately estimate the capacity anchorage details utilizing embedded bars.

### **Summary and Conclusions**

A lack of published literature and codified design provisions pertaining to pile-to-pile cap connections under uplift loading, coupled with survey results from the NCSC, and an ever increasing number of designs facing uplift situations, indicates a need for further investigation of the performance of various uplift connection details. The goal of this work is to provide information on cost effective and safe alternatives for the pile-to-pile cap connection. In order to do this, twenty-one full scale steel H-piles were embedded in simulated pile caps and subjected to tensile forces replicating uplift conditions in bridge substructure systems. A variety of anchorage types were tested with the pile embedded 12” into the footing, as well as bare piles embedded both 12” and 24”. From the investigation, it can be concluded that:

- 1.) For bare piles embedded 12”, the size of the pile (HP10 or HP14) did not have an impact on the ultimate capacity of the connection. Despite several NCSC respondents indicating that their respective agencies do not allow uplift at all, six tests performed with this configuration indicate that nominal ultimate capacities of 104 to 127 kips can be expected. Current methods for estimating bond by the Iowa DOT underestimate the connection capacity by an average of 190% for HP14’s and 275% for HP10’s. It is believed that the underestimation is the result of an incompatibility of the design methodology with the actual observed failure mode.
- 2.) Failure modes observed from testing bare piles embedded 12” indicate that concrete cracking leads to loss of bond between the pile and concrete.

- 3.) The results from the threaded rod and cage with bar specimens support published conclusions that C channels, angles, cap plates, and bar grids seem to produce stress concentrations near the tip of the pile which can lead to lower ultimate capacities than connections without such apparatuses.
- 4.) Utilizing V and U-bar anchor configurations provides a rational design approach because the capacity of the connection can be calculated from the area of embedded reinforcing steel. The capacity of connections utilizing reinforcing steel extending into the footing have higher capacities than other types of connections.
- 5.) Torching holes (as opposed to drilling) does not appear to have a significant effect on the overall capacity of the connection. However, size of the hole and level placement in the web of the pile is critical to achieving full capacity.

### **Implementation**

It is recognized that across state agencies design methodologies, construction practice, and loading conditions vary, sometimes substantially. Despite this fact, the applicability of this research is clear and the direct implementation of the results, combined with a good working knowledge of individual agency practice, provide safe and cost effective solutions to an oft overlooked portion of bridge substructure design. Therefore it is recommended that the following be considered:

- 1.) For bare piles embedded 12” into the footing, design should not follow previous design methodologies that assume a 0.060 ksi bond strength between the pile and concrete. Rather, a nominal resistance of 100 kips should be used for HP10s and larger. Resistance factors should be taken as:

$$\phi = 0.25 \text{ (for strength limit states)}$$

$$\phi = 0.40 \text{ (for extreme limit states)}$$

- 2.) When the nominal resistance of 100 kips provided by 12" of pile embedment is not sufficient, anchorage should consist of V-bars, U-bars, or by increasing the embedment depth to 24". When installing V or U-bars, details should follow the dimensions and layout provided in Figure 3. Holes can be torched or drilled ¼" larger than the nominal bar diameter. Hole size must be closely controlled during construction. For standard V-bar connections using #8 bars, a nominal resistance of 87.5 kips for each V-bar shall be assumed (totaling 175 kips for two bars), which incorporates the following resistance factors:

$$\phi = 0.25 \text{ (for strength limit states)}$$

$$\phi = 0.40 \text{ (for extreme limit states)}$$

- 3.) The standard details for U-bars using #6 bars should be designed with a nominal resistance of 50 kips per U-bar (totaling 200 kips for four bars). Development length must be achieved in these connections if the nominal capacities put forth are to be used. If 24" embedment is utilized, a nominal resistance of 175 kips should be used. Designers must also ensure that the bottom mat reinforcement in both directions is detailed so as to clear the piles by 1.5-2" on all sides. Additionally, the bottom mat reinforcement shall not be placed more than 13" above the bottom of the footing. Resistance factors for both the U-bar detail and 24" embedment shall follow the ones put forth for V-bars.
- 4.) When the 24" embedment detail is used, designers must ensure the depth of footing concrete above the tops of the piles is deep enough to prevent punching failure of pile through the top of the footing under compressive loads. If the minimum requirement is not met it is recommended that V or U-bars be used instead.

- 5.) Do not use anchor details that that incorporate C-channels, angles, clips, cap plates, bar grids, and other shallow reinforcement where the primary resisting mechanism is bearing and not development length.

### **CHAPTER 3. EXPLORING ACCELERATION COSTS ON TRADITIONAL BRIDGE PROJECTS IN IOWA**

#### **Introduction**

With current traffic demands on public infrastructure at an all-time high and projected demands only set to rise, maintenance on roadways is becoming increasingly difficult (Brookings Institute 2004, Cambridge 2005). To further complicate matters, the latest American Society of Civil Engineers (ASCE) Infrastructure Report Card issued the nation's infrastructure an overall D+ grade in 2017, with the bridge category achieving a C+ (<https://www.infrastructurereportcard.org>). In Iowa bridges received a D+ grade. The report states that although Iowa ranks fifth in terms of the number of bridges in service (approximately 24,000), "one in every five is rated structurally deficient, the third worst rating in the nation". Moreover, one-third of the total number of bridges are over fifty years old, with an additional 500 bridges reaching their fiftieth birthday every five to ten years (ASCE 2015).

In light of this, rapid completion of maintenance, rehabilitation, and replacement projects has proven crucial and has the following benefits (FHWA 2017a): reduced user costs, minimized traffic disruptions, increased safety of workers, shortens onsite construction time, and can reduce the environmental impact. Encompassing all of these values is the Iowa DOT's mission statement "Getting you there safely, efficiently, and conveniently." This core vision statement is one that will resonate with many state agencies, and it points to the fact that the rapid completion of projects can have a tremendous positive impact on both the traveling public and businesses. Any acceleration therefore, whether it be on the order of months or just a few days or weeks, can and should be viewed as a success.

Accelerated Bridge Construction (ABC) is one tool that has gained prominence in the last

ten to fifteen years, both nationally and in Iowa, as agencies have focused more and more on reducing impacts to the travelling public (Abu-Hawash et al. 2009, Nelson 2014a, Nelson 2014b). These types of projects often come with a high premium attached though, and are not suitable for all locations and/or constraints associated with a project. The Transportation Research Board *ABC Toolkit* (TRB 2013) acknowledges this, stating that ABC techniques are often associated with high initial costs and require more specialized equipment and qualified contractors to perform the work, thus deterring some State Highway Agencies from taking action and implementing these practices. A two-part paper investigating the construction cost of bridges using ABC and traditional techniques also observed a significant difference in the cost/ft<sup>2</sup> of bridges, with ABC being higher (Orabi et al. 2016, Hadi et al. 2016). Beyond the high cost and possible alienation of viable contractors to bid on projects, the relative newness of the design and construction methods mandate that they be monitored to see how bridges fabricated using these techniques perform under actual loading and environmental conditions. Current owners and agencies have stated concerns about the long-term durability of the precast elements, specifically at the joints and connections (TRB 2013). At least one paper investigating bridges using Prefabricated Bridge Elements and Systems (PBES) concluded that after only two years of exposure to live traffic and environmental conditions, non-composite behavior was observed near the abutments (RITA 2013). As of 2014, twelve projects in the state of Iowa were completed using ABC techniques, highlighting the relative newness of the technology and the application extent up to that point (Nelson 2014a). That number is expected to go up, however, as the expertise, materials, and contractor experience go up and the cost of these technologies decrease. While ABC techniques have their place in the industry, the feasibility and cost associated with these

methods as well as their limited application up to this point in time does not address the vast majority of bridges in an agency's network – of which the need for increased speed, reduced road closure time and user costs, is also of vital importance.

Besides utilizing ABC techniques, staged construction with no detours is another possible solution to minimizing user cost impact. The Iowa DOT currently estimates the cost of this type of construction as 10% more than the cost of a normal, detoured bridge project, per chapter 3 of the Iowa Bridge Design Manual (OBS 2017). While maintaining open lanes during construction is a viable alternative to accelerating a project to reduce user impact, it increases worker risk, can actually *increase* the overall project time, and can still significantly affect traffic depending on the extent of the partial road/lane closure.

For projects where acceleration is desired, but not to the point of using explicit ABC techniques (Slide-In construction, PBES, Geosynthetic Reinforced Soil – Integrated Bridge System, etc.), a third alternative solution is to “compress” the project schedule of a traditional project. A possible benefit of utilizing this approach over ABC techniques is that a traditional deck may be used, which does not include extra joints that are common when using modularized ABC methods. There is not a clear consensus in the industry at this point in time, but these joints may lead to the premature degradation of superstructure components through the infiltration of salt de-icers.

The compression of a traditional project can be done in a variety of ways including increasing crew size, working nights and/or weekends (overtime), utilizing extra or more advanced equipment, adding subcontractors, or re-sequencing the project (Al-Gahtani 2013). The cost associated with this “compression” activity is difficult to quantify, as noted by Loots in his exposition of construction law, “There is no hard and fast formula for calculating



acceleration costs” (cited from Maritz and Schutte 2009). While the cost of ABC materials has been investigated more extensively and can be linked to overall project durations, there is little relevant data available to speak into the relationship of cost and time in compressed traditional projects. Accounting for all project variables, Figure 3.1 shows hypothesized relationships between cost and time for a typical bridge project.

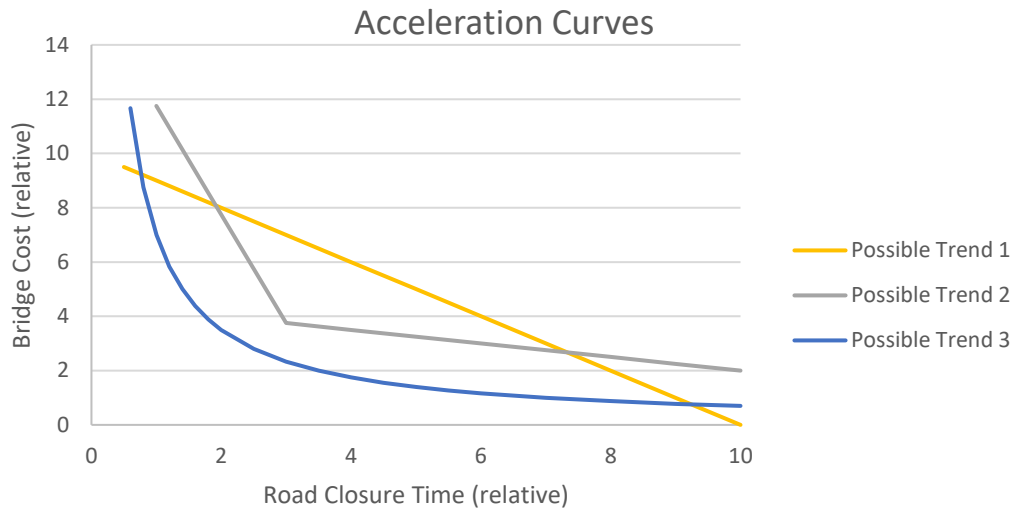


Figure 3.1 *Possible cost trends as contract time is decreased (i.e. projects are accelerated)*

Common sense states that the premium price for contractors to complete projects will increase as the time is decreased, but the extent of that increase and the relationship between the two are still unknowns. Besides the obvious user cost savings associated with compressing a project schedule, there are many other reasons why entities might want to alter a schedule. These include permitting/seasonal work restrictions, planning around specific/local events, railroad and Right-of-Way considerations, and specific site constraints, among others. The factors listed all have direct time-cost trade-off behavior, and as such, in order to properly evaluate these considerations agencies could benefit immensely from a better understanding of this relationship. The goal of this research is to establish guidance

that contracting agencies could use to accurately estimate the cost of bridge projects as contract durations are manipulated. This is a tall order, however, and there is little groundwork that has already been laid in the research area. As such, the focus of this research is to begin exploring the relationship between cost and road closure durations for replacement bridge projects in Iowa and to provide a framework for taking concrete steps forward in this topic. To do this a thorough review of literature, along with historical data analysis from the Iowa DOT and personal interviews with contractors and DOT personal were conducted.

### **Development of contract durations in Iowa**

In order to more fully understand the existing relationship between cost and road closure time for bridge projects, a detailed look into how the Iowa DOT determines contract periods was necessary. To do this, the Letting Guidelines were used as a reference in conjunction with the Federal Highway Administration (FHWA) TA 5080.15 ‘Guide for Construction Contract Time Determination Procedures’ (OC 2017, FHWA 2002). Meetings with Iowa DOT personal were also conducted in order familiarize the research team with the process of setting contract durations and road closure times for projects.

While flexibility in setting contract durations is stated as a value in the Letting Guidelines (more flexibility allows the contractor to more efficiently schedule their work, crews, and equipment in an efficient manner, which should decrease the cost), the following five step methodology which prioritizes accuracy over flexibility is also laid out:

- Step 1: Become familiar with Traffic Control Plan, Staging Plan, items of work and estimate reference notes

- Step 2: Determine which types of work will most likely control the schedule and where overlap may exist
- Step 3: Use the “Average Production Rates” chart provided in the Appendix of the Letting Guidelines to estimate working days for controlling work types
- Step 4: Take into consideration any special circumstances included in the project and adjust accordingly
- Step 5: Estimate total number of working days for project; round up to nearest 5 days

Paramount to quantifying the acceleration costs associated with compressing a project is understanding how much “flexibility” is currently built into the projects. For example, a project with a high degree of flexibility already built in will most likely not cost much to accelerate, up to a certain point. Once the critical point is reached where more crews, equipment, and/or materials are needed, or the schedule needs to be modified, costs will begin to rise. The methodology laid out above assumes that the contract durations are close to the actual required time to complete a project and there is little, if any, flexibility. This is of course dependent on how accurately the contractor’s speed for various tasks can be estimated.

When asked if the Office of Contracts adheres to the process laid out in the letting guidelines, it was discovered that they “loosely” do. While efforts are taken to conform to the five step process, often there are other considerations that must be taken into account, as noted in Step 4. This usually involves working with the Resident Construction Engineer (RCE) for the district where the project is located, who is more familiar with the project constraints and considerations. Setting contract durations then becomes an iterative process, where factors within and even outside of the physical project – such as local events or

traffic/detour considerations – can play a role in the final timeframe that is assigned for a given project.

Following the procedures given by the DOT, it was concluded that for most projects the contract duration set is a reasonable time to complete the work required for a bridge replacement project. Practically, contracts are generally not let with a high degree of flexibility already built in, as suggested as a value by the Letting Guidelines. Instead, they aim for an accurate timeframe in which to realistically finish a given project. Questions still remain as to the accuracy of this estimation and the contractor's perception of given timeframes in which to complete the work. Additionally, it was suggested that a minority of projects over the last number of years may have been intentionally accelerated due project constraints. As such, a more in-depth review of historical data was needed to verify and examine these, as well as to review the Iowa DOT's Office of Contracts letting methodology.

### **Overview of Acceleration**

Accelerating construction projects (also known as compressing or crashing a project) in theory is fairly straightforward. However, once applied to actual projects with special considerations and/or irregularities, as is the case with many bridge replacement projects, the ability to estimate durations and costs becomes notably more complicated. In their analysis of the suitability of ABC implementation and the decision-making process, Salem & Miller (2006) note that both “qualitative (e.g. safety of motorists and construction workers, impact on surrounding communities and businesses, environmental impact) as well as quantitative factors (costs, impact of traffic flow) exist.” In the case of traditional contract length projects, these same factors must be evaluated to determine the appropriate amount of acceleration. The following is a summary of typical project costs noted in relevant literature, methods to accelerate projects, and the theoretical approach to estimating the additional cost

associated with acceleration.

Epstein and Maltzman (2013) describe the five typical types of costs that are incurred on a project in their book, *Project Management Workflow, A Business Process Approach*: direct, indirect, fixed, variable, and sunk. In addition to the cost factors, Al-Gahtani (2013) notes that the additional risk incurred by acceleration - due to the increased work on the critical path and less overall float time - adds to the project costs as well and should not be neglected.

Efforts to pinpoint specific tasks or activities that contribute significantly to the cost of acceleration, as well as the ability to accelerate, were not identified from a review of current literature. This is most likely due to the uniqueness of projects and their subsequent constraints. However, general methods to reduce activity durations and accelerate projects are proposed by several authors (Al-Gahtani 2013, Elbeltagi 2009, Ness 2010):

- Applying multiple-shifts work
- Working overtime/weekends and holidays
- Offering incentive payments to increase the productivity
- Using additional resources/equipment
- Using materials with faster installation methods
- Resequencing or rescheduling the project, fast-tracking (Epstein)

In order to reduce project durations and find the optimal time with the least cost, the most common method used in industry is Time-Cost Trade-Offs. The method assumes that as a project duration decreases, indirect costs decrease and direct costs increase (shown in Figure 3.2). For the purposes of simplification, the other costs noted by Epstein are lumped into either indirect or direct costs.

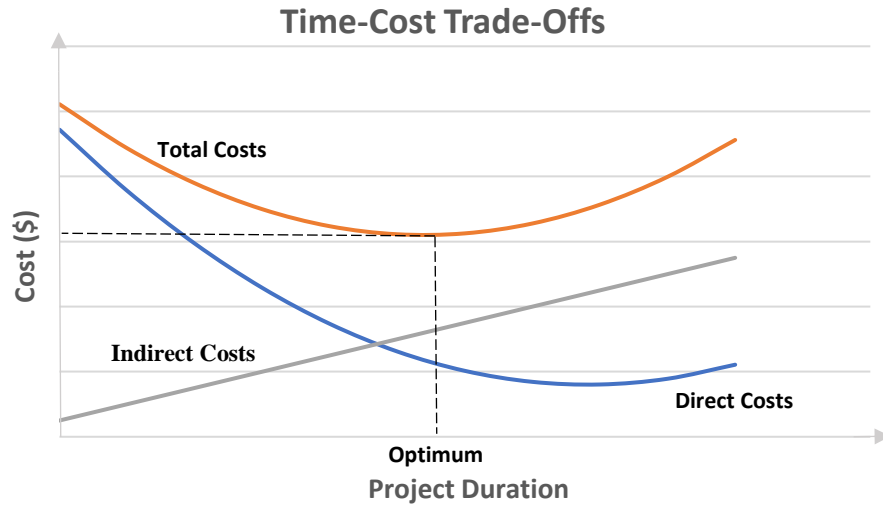


Figure 3.2 *Time-cost trade-offs*

Total project cost is then the sum of both direct and indirect costs. Because of the inverse relationship there is an optimum point, as shown in the figure. To use this method, acceleration of *each* activity on the critical path is analyzed through a simple formula:

$$\frac{(\text{Crash Cost} - \text{Normal Cost})}{(\text{Normal Time} - \text{Crash Time})} = \text{Cost Slope}$$

The relationship between each activity time and cost is assumed to be linear.

Activities with the lowest cost slope will be reduced first along the critical path, as that reduction in time will result in the lowest added overall cost to the project. It should be noted that as the total project duration is decreased, evaluation of possible new critical paths need to be performed.

While the Time-Cost Trade-Off methodology is a good tool to evaluate the optimum project duration, this assumes that one knows the crash time and the associated crash cost that goes with it, which is the focus of the research. Previous research seems to suggest that the only way to quantify acceleration costs is to look at projects on an activity by activity basis, as the Time-Cost Trade-Off approach takes. Such analysis is not only tedious and time

consuming, but extremely hard for estimators to perform. The Florida DOT makes a step in this direction by utilizing production charts with low, average, and high production values for different activities (FDOT 2010), thus giving an indication of how fast a contractor could perform work. The basis for arriving at these general production values is unknown however, and would have to be modified to fit the capabilities of the construction firms in question, which may vary depending on location. Additionally, there is still no metric to measure the cost associated with the differing levels of production.

### **Historical Data Analysis**

The first approach taken in this research to investigate acceleration costs was to analyze bridge projects in Iowa from a “bird’s eye view”. To do this, information was collected on 285 bridge construction projects from 2012-2016 from the Iowa DOT’s database. Plan sheets, bid tabs, and letting documents were consulted when necessary to gain more information on specific projects. Because of the complex nature of bridge projects, any available and relevant data were included in the analysis as shown.

Table 3.1 *Project constraints included in analysis*

Contract Type	Contract Award Amount and Duration	Letting and Start Dates	Bridge Type
Contracting Authority	Location	Contractor	Deck Dimensions
Route	Road System	Feature Crossed	Detour Information

Pertaining to the ‘time’ criteria, no data were accessible concerning the actual road closure times for specific projects. In lieu of this, overall contract duration data were used. From these inputs given above, it was hypothesized that bridge type, size, road system type, and the inclusion of detours would have a large impact on the ability and cost of acceleration.

One of the charts showing the initial breakdown of this data is below in Figure 3.3. For reference, the “Other” bridge classification encompasses precast panel, modular steel beam, slab, and other non-traditional types of bridges and are used almost exclusively on small bridges.

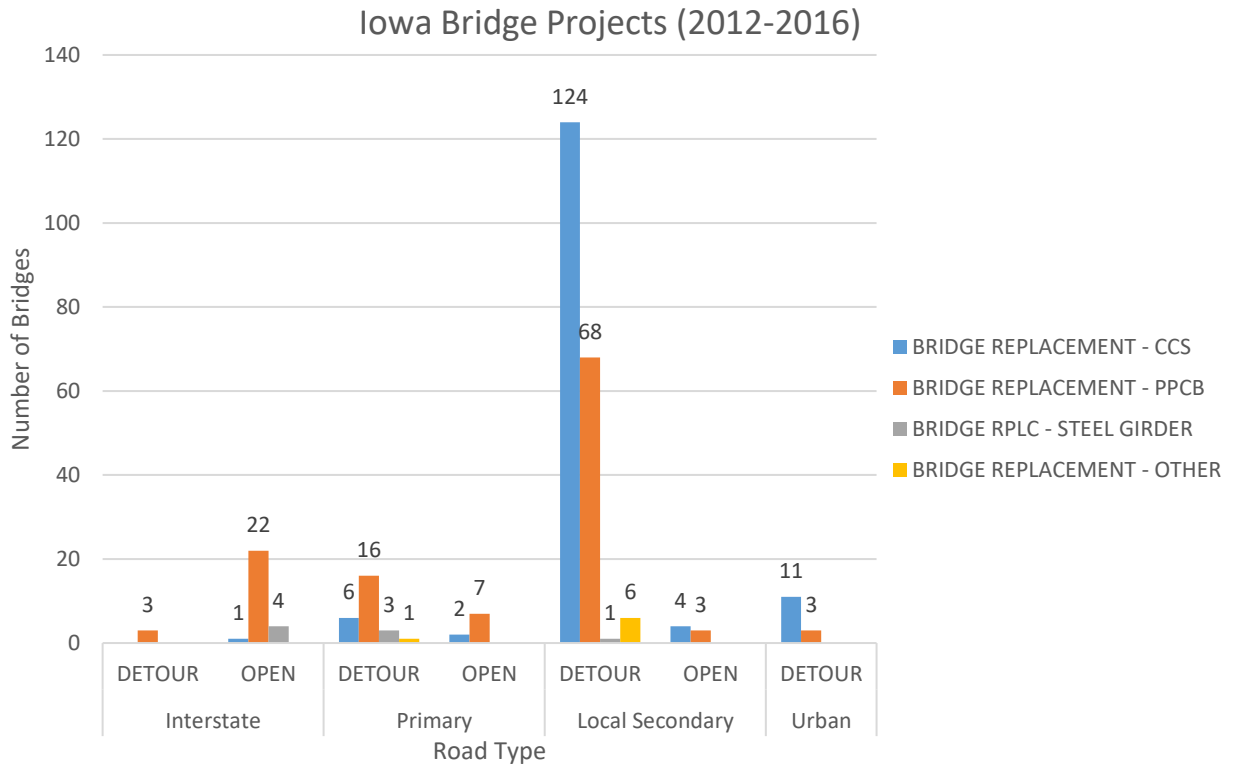


Figure 3.3 *Iowa Bridge projects by road type, material, and specified closure*

Initial attempts to identify the relationship between total project cost (e.g. bridge substructure and superstructure, earthwork, approaches, etc.) and contract time necessitated accounting for project specifics that might contribute to information embedded within the data. Size of the project was therefore considered by converting the total project cost to a dollars/ft<sup>2</sup>. Using these data, cost/ft<sup>2</sup> vs. contract time were then plotted according to bridge type and road system. Figure 3.4, which defines the data by bridge type, is shown below. No recognizable trends as hypothesized in Figure 3.1 were observed; rather, the charts are characterized by the largest concentration of projects in the 50-120 day contract time and



associated costs varying from \$60-200/ft<sup>2</sup>. This leads to the conclusion that other factors are affecting the cost and/or time aspect. Plots comparing bridge cost to size also confirmed this fact, as in some cases similar sized bridges could vary in price by as much as 300%. It should be noted that all projects above 200 days were associated with contracts that contained more than one project (e.g. other bridge, paving, lighting, etc. projects were included- thus the contract time cannot be directly tied to specific bridge project shown).

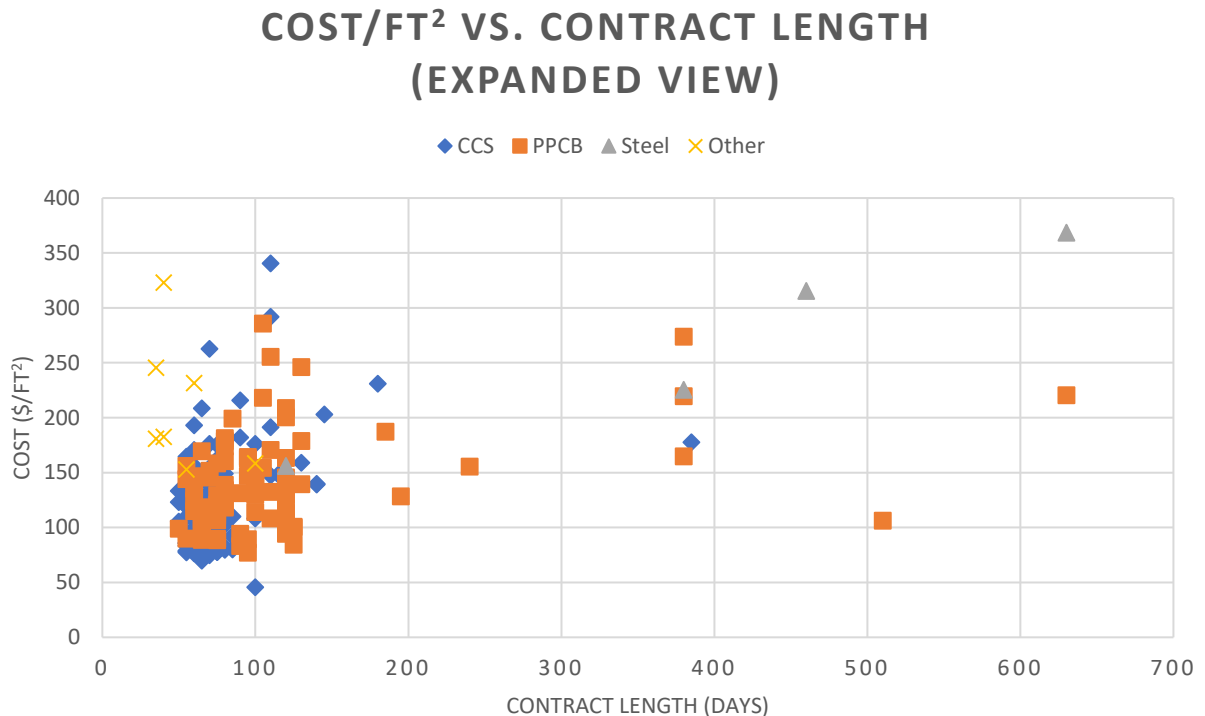


Figure 3.4 *Cost/ft<sup>2</sup> vs. Contract Length based on material type*

While no relationship quantifying the acceleration costs can be drawn from the above data, several general observations can be made. First, bridge types falling under the “Other” category (which include precast deck panel, modular steel, and slab bridges) are, as a group, higher in cost than PPCB and CCS, indicative of the different construction techniques associated with ABC techniques. This correlation confirms findings from both Orabi (2016) and TRB (2013). As shown by four of the seven total projects, these non-traditional types of

bridges are usually used when project constraints dictate a shortened contract time (i.e., 35-40 days). No CCS, PPCB, or steel girder bridge was let with less than 50 working days in the contract. This does not mean, however, that traditional bridges cannot be completed in a shorter timeframe- it simply indicates that attempts have not yet been made to reduce them below the 50 day mark. The feasibility of this shortening is directly tied to the scope of the project and critical path, as noted by Al-Gantani (2013). Because the data consists predominantly of either CCS or PPCB bridge projects, a more in-depth breakdown of the data is shown below in Table 3.2. From it, we see:

- CCS bridges are, on average, less expensive to fabricate than PPCB and require fewer working days. (note that this may be because they're generally associated with smaller projects and the foundation work is usually simpler)
- Bridges on primary or urban roads are more expensive than bridges on the local secondary system

Table 3.2 *Cost and contract information on CCS and PPCB bridges*

		No. of	Average	Cost (\$/ft²)		
		Projects	Working Days	Average	High	Low
CCS	Local					
	Secondary	119	68	\$116	\$340	\$70
	Primary	5	97	\$195	\$292	\$96
	Urban	8	71	\$148	\$176	\$104
PPCB	Local					
	Secondary	64	87	\$131	\$246	\$77
	Primary	9	113	\$174	\$255	\$132
	Urban	2	113	\$200	\$286	\$113

\*Excludes all projects with 'Open' category, interstate roads, and contracts that were let with multiple projects and/or special contractual language (e.g. A + B bidding, I/D, major intermediate milestones, etc.)

In addition to this, to gain a better understanding for the costs and scope of the work on similar sized projects and perhaps normalize the data, bid tabs were pulled for CCS and

PPCB projects of with approximately the same deck area and compared. Unsurprisingly, the material cost of the bridge (namely the structural concrete, piling, steel, beams, etc.) were significant portions of the project cost for all bridge projects. However, there were several differences that emerged between projects with higher cost/ft<sup>2</sup> than the low dollar projects. Common activities and items are highlighted in the Table 3.3.

Table 3.3 *Bid activities existing on bridge projects with high cost/ft<sup>2</sup>*

<b>Higher-end \$/ft<sup>2</sup></b>
High mobilization costs
Large volume of excavation*
Revetment installation
Large embankment installation*
Aesthetics
Complex bridge removal*
Railroad ROW involvement
PCC paved deck
Approaches included
Rock coring
Scope large (> 50 items of work)

\*Indicates an activity that could contribute significantly to overall contract duration

Projects that were let with the same contract duration were also compared, specifically those at 70, 110, and 120 days. All projects with a 70 day contract length had a similar scope of work (15-20 items) and can best be described as a basic bridge, in which costs ranged from \$70-150/ft<sup>2</sup>. The exception to this was bridge #246, which had 68 work items including paving, high mobilization costs, high revetment, and high cost of the removal of the existing bridge, resulting in a much higher unit cost of \$263/ft<sup>2</sup>. Despite the increased scope, the contract duration was still 70 days. Project costs could have increased due to a) increased scope, b) accelerated schedule, c) route is a primary road, or d) any combination of the above. There was not enough conclusive evidence in the rest of the data to determine which

of these factors affected the cost. Other general trends observed:

- As contract duration increases, so does the number of work items on a given project
- More significant embankment and excavation were noted on projects with 110 and 120 days whereas 70 day projects had very minimal embankment and excavation needed

Given the uniqueness of bridge projects and their constraints, it was impossible to normalize the data to an “ideal” project cost and time. In fact, while higher dollar projects (\$/ft<sup>2</sup>) usually (not always) included more items of work and a longer contract duration, this was not always the case. From the in-depth analysis of the projects, it appears that the Iowa DOT letting guidelines criteria for arriving at a contract duration (using production charts) is generally followed. However, there do exist projects of varying scope that are let with similar contract durations. The reasons for this deviation from the common observed trend between scope and time on projects is unclear and the data were not conclusive.

### **Contracts and Project Time**

While extra time or flexibility is not included on projects from the contracting agency’s perspective, a natural question that arises is ‘How much flexibility do contractors feel that they have?’ As noted earlier, understanding this can help answer the question of when acceleration costs will start to increase rapidly as days are decreased on a project. Information on the site time charged vs. site time allowed was gathered on 83 completed bridge projects in Iowa over the past 8 years. From this it was determined that 60% of projects were completed under or on time, while 40% incurred project overruns. Of the 60% that were completed within the contractual guidelines, just over 25% finished 18 days early or more, indicating some possible flexibility. However, of the projects that exceeded the

contract length, 33% finished 18 days or more behind schedule. The large range in projects over/under time gives an indication of difficulties associated with estimating the contract duration. Weather delays, improper estimation by the contracting agency, contractor management, resource availability, among many other reasons may be the cause of the discrepancy. Trends were certainly observed by evaluating the on-time delivery of the individual contractors themselves, with some completing all or a majority of their projects on time, and others consistently going over. At the very least, the data show that under the right circumstances, contractors are able (and even willing) to accelerate projects.

Contract formation can also play a key role in accelerating bridge projects. Most of Iowa's projects are let with a specified 'Late Start Date' and use working days, which follows a 'flexible' methodology. This would generally give the contractor the maximum amount of time in order to schedule crews and work before the project has commenced. In light of industry demands for more rapid completion of projects and the need for projects to be completed on time, alternative contracting has become more and more common, not only in Iowa but around the country (FHWA 2017b). A further look into the specific contracts that each of the 285 projects were let under revealed a not insignificant number that included special considerations, as highlighted in Table 3.4 below.

Table 3.4 *Projects let under contracts with unique considerations*

<b>Road System</b>	<b>* Projects</b>	<b>Total Projects</b>	<b>Percentage</b>
<b>Interstate</b>	27	30	90%
<b>Primary</b>	19	35	54%
<b>Urban</b>	3	14	21%
<b>Local Secondary</b>	9	206	4%
<b>All</b>	58	285	20%

\*Projects with contracts that have special stipulations (i.e. I/D, A + B, No Excuse Road Opening Bonus, major intermediate milestones, or staged construction)

No excuse road opening bonuses, major intermediate milestones, calendar completion date, and specifying critical closures and/or staged construction have been used where project constraints necessitate completion by a certain date (OC 2017). Acceleration is not the goal when contracts of this nature are let, but rather these highlights efforts on the contracting agency's part to incentivize and ensure completion by a designated timeframe- an important distinction. Intermediate milestones included in the contract were found to be associated with several higher than average cost/ft<sup>2</sup> bridges. This does not imply causation, and could simply be a correlation in the data but it nevertheless warrants pointing out. As seen in the Table 3.4, the more heavily trafficked arterials (interstate and primary roads) are more apt to use alternative contracting. Interstate projects in particular are often unique and need to be considered individually- not only because of the added stipulations, staging requirements, and milestones, but also because these contracts generally contain 4-10 projects that are governed by a single contract duration (with intermediate project dates). Attempts to make accurate inferences from this data onto other projects is nearly impossible. Contracts that are let with a number of projects may 'skew' the cost of each individual bridge project. The Iowa DOT Letting Guidelines point out this distinction and associate multiple-project contracts with lower unit costs (OC 2017). There was insubstantial evidence in the historical data to either support or negate this claim.

In rare instances where the agencies do attempt to accelerate, the two most common types of contractual mechanisms to accomplish this are Incentive/Disincentive (I/D) and A + B bidding. Of the 285 projects analyzed, fourteen utilized I/D and three were let under A + B bidding regarding the bridge work. Pertaining to the I/D projects, all but one of them were let with other projects (such as paving, lighting, signage, culverts, etc.), which drastically

increased the contract time. Because of this, comparisons attempting to quantify acceleration costs between bridge projects of similar size and scope for traditional (non- I/D) and I/D projects could not be made.

A + B bidding is contract arrangement in which contractors add the direct project costs (normal bid) to the calculated user cost (e.g. \$5,000/day) based on the number of days they bid to determine the total A + B bid amount. In this manner the acceleration of a project is incentivized and a contractor is able to choose a contract duration that best suits his/her own availability and resources. By taking a reduction in the contract time the contractor can allocate more resources/spending to actual bid items (which will usually go towards increased mobilization costs and added crews to complete the project in the faster specified timeframe). On the three projects that utilized A + B, user costs were set at \$2,500/day, \$3,000/day, and \$10,000/ day. Surprisingly on all three projects, all of the contractors that submitted bids selected the maximum number of days allowed for the contract, and none chose to accelerate. Even though three projects is a small sample size, the fact that *all* of the contractors – not just the ones that ultimately won the project – chose more days over cutting time is significant. Results may have been different if contracting agency incentivized acceleration more highly.

While examining historical data did provide some interesting and useful insights, a specific relationship between bridge projects pertaining to cost and contract duration could not be ascertained. For the majority of the bridge projects it was determined that acceleration has not been attempted, and has in fact only used when I/D or A + B language is specified in the contract. Because of the complexity and large scope of these accelerated projects, parallel behavior to the projects under ‘normal’ durations could not be extrapolated from the data. A

larger pool of data analysis would prove beneficial only if: 1) singular bridge projects were let using I/D; 2) A + B bid time was less than the maximum specified amount; or 3) projects without any additional contractual language were known to have been accelerated.

### **Contractor Perspective for Bridge Acceleration**

Much of the current literature on accelerating projects is based on a more theoretical approach, which unfortunately leaves out the contractor's ability to voice their own input and concerns into the feasibility and cost of acceleration. To help shed light on this topic, three different sized construction firms that operate in Iowa and the Midwest and have done recent bridge projects were solicited for personal interviews. For simplicity and confidentiality, these firms will be henceforth be called Contractor A, B, and C. The format of the interview consisted of a series of general questions about the firm itself, construction practice, acceleration, and questions pertaining to a case study.

Capturing a range of contractor perspectives was key when conducting the interviews, and as mentioned three distinctly different firms were selected. Their differences in size, work expertise, types and number of projects awarded, among other details were realized in order to more accurately interpret the data from the interviews. Qualitative descriptions of the contractors can be seen below.

#### *Contractor A:*

This was an example of a large firm, with a total of 20-25 crews during the construction season. Number of projects awarded per year is approximately 30, with the work broken down roughly into 'New Construction/Structures' (60%), 'Bridge Deck Overlay' (30%), and 'Rehabilitation' (10%). The breakdown of projects can vary from year to year, and it was expressed that not many years ago a large number of the projects came



from bridge deck overlays. This demonstrates the flexibility and resource allocation availability of a firm of this size. Contractor A has done work out-of-state as well as in-state, and has been known to bid on larger, more complex projects.

*Contractor B:*

Can best be described as a middle-sized firm with associated capabilities. With around five crews, they are able to complete approximately 5-10 projects each calendar year. Most of their work (75%) is done on 'New Bridge' or Replacement Bridge' projects, while the remaining 25% encompasses 'Deck Replacement', 'Deck Repair', 'Culverts', and other smaller projects. The type of work depends on the season and projects available, but recently they have seen an uptick in bridge replacement projects. Awarded projects have varied from larger and more complex to relatively simple.

*Contractor C:*

Is most appropriately categorized as a smaller contracting firm in terms of capability. Their self-described niche would be small, generally county CCS slab bridges of the 80'-150' long variety. Roughly 30% of the work that they do is performed for the Iowa DOT, with the remaining 70% is owned by counties or cities. 80% of their work is done on 'Replacement Bridges', with the remaining 20% allocated equally between 'New Bridge' and 'Deck Repair/Replacement'. They have 5-10 crews with an estimated 5-6 people per crew. Because of this focus area and the often small scale of projects they are able to complete 20-25 a year.

### **General Questions**

Throughout the course of question and answer portion of the interviews, common themes and differing opinions on certain aspects of accelerating bridge projects became apparent. They will be discussed in further detail in this section of the paper.

### **Flexibility in Current Contract Durations**

When asked about the current flexibility and ability to efficiently schedule crews to certain projects, Contractors A and C had remarkably different answers than Contractor B. Contractor A felt like there was sufficient room to accelerate most bridge projects, but in their view would rather use traditional construction methods and materials and work multiple crews 24 hours/7 days a week in order to complete projects as opposed to introducing new accelerated bridge techniques. They even went so far as to say, “We have not seen how fast we can go yet.”

Contractor C also felt that there was adequate time given in most contracts in order to efficiently schedule their crews as well, stating that they completed 90% of their work on or before the specified number of days. For a 125' CCS bridge (a staple of Contractor C's project/revenue stream), the ideal window would be around 60 days. It is important to note that the size and scale of the projects that are undertaken by Contractor C is dramatically different than those awarded to Contractor A. The ability of Contractor A to efficiently manage their projects may be connected to their experience, large resource availability, and capability to handle diverse projects. Contractor C's success and perceived relative flexibility in the contract duration can most likely be tied to the fact that they serve a specific niche of the industry, and as such have become very proficient and know where to put their people and resources in order to produce successful projects.

Conversely, Contractor B responded to the same question of acceleration by stating, “We already feel like we're in a box.” For most bridge projects they felt that the given contract duration was already a tight window, emphasizing that they generally use every day in the contract. To them, 90-120 days is the sweet spot and shortening project times below that was undesirable. To be fair, very few of Contractor B's projects are of the same variety

and scope as Contractor C's – if they were then they would likely agree that the 60 day timeframe was adequate. Intermediate milestones specified in the contract documents also added to the 'constricting' feeling and consequently increased the risk. The correlation noted between higher \$/ft<sup>2</sup> projects and the presence of intermediate milestones in the historical data may be a result of additional risk highlighted by Contractor B here.

### **Critical Path**

Critical path activities were also a subject of inquiry, as a proper grasp of them can lead to accelerated and efficient, yet realistic contract durations. Earthwork was a common activity mentioned by all three contractors as directly impacting the scope and schedule of a project. It is important to note that it is not only the volume of earthwork that is necessary to consider, but also the type and the weather during this stage of construction. While reiterated by the contractors, this fact was also noted in the historical data analysis. Earthwork removal ranged from \$4/cy to upwards of \$350/cy depending on the type. Additionally, projects with high amounts of earthwork (specifically embankment-in-place or excavation) tended to have much longer contract durations than those that had little earthwork. Substructure work, namely type of foundation and method of installation, is also an important construction activity with a wide range of possible days depending on the methods employed. Contractor A highlighted the obstacles to utilizing drilled shafts as opposed to piling: specialty consultants are usually required, the installation is costly, and it can add weeks or even months to a project's schedule. If drilled shafts are specified in the plans, Contractor A said they would often try to 'Value Engineer' (VE) the item, which means that they will provide an alternative that provides greater value to the owner (in this case the Iowa DOT). Contractor B noted that even without drilled shafts, piling was a significant activity on the critical path.

Naturally concrete cure times were on the critical path, although they seemed to affect the contractors to different degrees. When it comes to acceleration, especially for projects utilizing traditional methods of construction, this has been the proverbial wall that needs to be overcome. Concrete curing durations and sufficient strength are specified by the governing entity's specifications, and can often be found on the critical path. However, Contractor A noted that with higher strength concrete and that by employing maturity curve testing to meet the minimum strength specifications rather than waiting for the minimum number of days, they are able to cut down drastically on these activities. As a result, and with the appropriate resources and equipment, the overall schedule time for the project can be significantly shortened without using ABC techniques. Contractor C also employed maturity curve testing with great success, although they did describe problems occasionally with private engineering firm inspectors allowing the practice over the 'time duration' specification. Contractor B, while mentioning the applicability and possibility of time schedule savings of maturity curve testing, did not employ it for the most part, which may contribute to the 'constricted' feeling that they have pertaining to project schedule. They too expressed some concerns with inspecting agencies accepting the practice and thus had to wait for the specified cure times.

In addition to these activities, aesthetics, number of sub-contractors, Right of Way (ROW), utility coordination and relocation, and lead times to procure PPCB beams were also noted as potentially adding larger time blocks to projects. Several of these latter items can (and often should) be completed or in the process before the contractor actually ever steps foot on the project site.

### **Acceleration/Innovation**

Feelings toward acceleration and innovation were mixed and generally followed

individual contractors' perception of the current extent of flexibility and maneuverability within the contracts. Both A and C were open to acceleration. Contractor A, as stated before, would rather utilize traditional methods of construction to accelerate projects (overtime, weekend/night work, adding crews, stacking activities, etc.) than employ ABC techniques. The contractor's hesitancy to use accelerated components in favor of traditional was tied to the additional risk assumed when using new and innovative building techniques. Contractor A stated that while utilizing new techniques was not preferential due to the additional risk, they were certainly open to them. They stressed that this increased risk is translated into substantial dollar cost to the owner though, which is to be expected. Contractor C was also open to acceleration and innovation, citing their ability to get current projects done on or before schedule. They stated "we will be the first" to try out new techniques where others might not be comfortable. Again, this quote must be taken within the context of the size and scope of the firm- they are willing to try innovative techniques on projects most likely within their niche. They supported this by bringing up several examples of new material/construction methods that have been used on a handful of the projects they've done in the last several years. Contractor B, while not opposed to acceleration or innovation if properly compensated, did not speak as confidently or enthusiastically at the prospect of shortening durations.

The polarity between these contractors cannot be understated. One possible reason for this is the size and availability of resources of the respective companies. Mobilizing resources may be much more difficult and expensive for a smaller sized firm. That is, if the projects are varied and diverse, as are the ones that Contractor B takes on, project costs will most likely be increased. Contractor C presumably has adequate means and methods to

procure resources for their typical niche projects, even if they are a smaller firm. Outside of their niche, it is conjectured that they would have similar feelings to Contractor B. Another reason for the discrepancy could be the experience level associated with constructing and managing complex projects, or those needing to be completed in a rapid timeframe.

A fear that was expressed with accelerating contracts is that contractors may be ‘penalized’ later if they show that they can complete projects more rapidly by having unnecessarily tight and restrictive durations that drive up their costs and cause them to lose revenue. Therefore, even if they are able to accelerate projects now and complete them early, often they will not. Proper compensation could erase this fear and promote acceleration that both the public and state agencies desire.

Previously some of the critical path activities were noted. Compression of the project schedule to accelerate can happen by providing more crews, resources, and equipment on a project to finish tasks quicker, or by ‘stacking’ work tasks that can be completed simultaneously. Interestingly, it was found that sometimes the ability of the contractor to accelerate a project has more to do with external factors than internal to the project itself. Perhaps the most frequent recurring item expressed outside of the contractor’s control (and affecting acceleration) was the quality of inspectors on projects. Even without acceleration, the contractors (especially B and C) noted the tremendous impact that a “good” inspector can have on a project in terms of both cost and schedule. At least one of the contractors has done an internal review of project revenue cost savings, and it was found that a substantial amount of savings or overruns were tied back to the quality of the inspector and relationship with contractor. Because of this it was expressed that for some inspectors overall project costs could be inflated up to 10%. Simply put, Contractor C stated “we appreciate the value of

good inspectors.” When accelerating projects then, the inspector has a huge responsibility and role in dictating whether the contractor stays on schedule or not.

Environmental considerations including mussels, birds, bats, and storm water runoff were also noted as potential activities that hindered acceleration. While the enforcement of the environmental considerations is mandatory and often they are impossible to avoid, contract durations where these factors are present should be adjusted accordingly.

Accelerating under these conditions, or others of a similar variety, would be very difficult to project. As expressed earlier, ROW and utility reallocation along with lead times for pre-stressed concrete beams will also play a role in acceleration. Increasing the time between the letting date and late start date may greatly aid the ability of contractors to accelerate.

### **Costs**

Pertaining to costs, all three contractors said that decreasing the contract duration would increase the expenditures due to increased overtime, equipment, and risk, albeit to seemingly different degrees. Overtime costs are allocated to contractors and can be calculated simply enough by taking the total number of man-hours required on a project and dividing it number of weeks and crew size. The amount of overtime required will most likely vary depending on the size of the construction firm (i.e. larger firms may utilize more crews rather than pay for overtime). Additionally the mobilization aspect, acquisition of additional equipment, and risk component are based more on the contractor’s set skills and abilities and can vary from firm to firm. As highlighted previously, these factors seem to adversely affect

Contractor B’s ability to accelerate more so than Contractors A and C, and are thus more costly overall for the former firm.

Contractor A noted three key areas that drive the price on a project: ability to do own

work (i.e. no sub-contractors or specialists), competition, and risk. The contractors all noted project markups when the feature crossed or carried was an interstate system. And lastly, while Contractor C was open to new construction methods they spoke of an approximate 50% cost increase to implement these innovative practices.

### **Case Study**

An example bridge project was presented to the contractors in order to ascertain what kinds of cost could be associated with acceleration. No calculations were asked to be performed; rather the goal of these inquiries was to establish a general relationship based on the contractors initial best guess.

The bridge replacement project presented was county road E-57 over the interstate. Project plans and letting documents may be found on the Iowa DOT's website under the project number: IMX-035-4(188)110--02-85 ([www.iowadot.gov/contracts/plans-and-estimation-proposals](http://www.iowadot.gov/contracts/plans-and-estimation-proposals)). The new bridge was 33' wide x 446' long, and included five spans and four piers. The bid amount was \$2.4 million (giving a slightly higher than average cost/ft<sup>2</sup> of \$163). The contract duration was 120 days, which encompassed two other projects. Work for the bridge was sub-contracted out (potentially adding to the higher cost/ft<sup>2</sup>) to a sole contractor who performed the work over the course of the contract.

Each contractor was told general project details, and then given the plans and bid tabs to look over briefly in order to become acquainted with the project. They were then given a chart (similar to Table 3.5) and asked to fill out the percentage change in cost assuming that at 120 days the cost was 'X'. Contract durations were varied to track the change in cost, as well as road closure time as the contract duration stayed constant. The original questionnaire included a 15 day slot, but all contractors responded that a time duration of that length would be impossible for the project if traditional methods were used and the alignment was the



same. Even if the contract duration was 120 days with the road closure time set at 15 days, the same opinion was voiced. Results are summarized in Table 5 with discussion following.

Table 3.5 *Case study results*

<b>Contract Duration/ Road Closure Time (Days)</b>	<b>Relative Cost</b>		
	<b>Contractor A</b>	<b>Contractor B</b>	<b>Contractor C</b>
(120/120)	X	X	X
(60/60)	1.1 X	1.4X	2X (or more)
(30/30)	1.2-1.25X	Impossible	Impossible
(120/60)	--	--	2X (or more)
120/30)	--	2X (if possible)	Impossible

Much of the results support what the contractors expressed in the beginning of the interviews. In addition to validating these claims, further explanation of practices, acceleration capabilities, and cost were gained through the follow-up dialogue. The presence of acceleration impacted Contractor A the least in terms of cost. Reducing the contract time by 75% down to 30 days would surprisingly only produce an approximate 25% increase in the original cost of the project. These would be mostly due to overtime and acquisition of additional equipment. Contractor B expressed the 120 day duration as the most feasible and economical for them. They were hesitant to reduce the project time to anything below the original time, but eventually decided that the premium price to go to 60 days would be around 140%, which would capture the added overtime, equipment, and management costs. Thirty days was out of the question according to Contractor B's staff. For a bridge of this size and magnitude, Contractor C was also hesitant to reduce the working days to anything below the initial 120 days. They admitted that while they have done a few projects with this scope, the bridge was larger than most and one that they would probably not bid on in an actual letting. Their initial cost would most likely be higher than both Contractor A and B's due to

the increased mobilization, as well as the added complexities involving crossing an interstate. When questioned if the project was achievable in 60 day timeframe, Contractor C was extremely reluctant to give an answer, but conceded if it was possible the cost would be at least 200%, if not more. They were not confident in their ability to do this and would not take on the project in a real-life scenario.

Crew size grew proportionally to the number of days cut for Contractors A and B. At the 120 day mark, these firms would have one crew working on the project. Decreasing the number of days to sixty would most likely necessitate the introduction of one more crew, and a further reduction to thirty days would double the crew size again, giving four total crews. In contrast, Contractor C would most likely have to allocate all of their resources to the project to complete it in the shortened 60 day window. Contractor's A and C noted that four crews on project was the maximum number that they would have (about 24-25 people), as the introduction of more crews after that would be impractical. They stated that productivity and efficiency even with three or four crews would be reduced when compared to just one or two crews. This fact is reiterated by Al-Gahtani (2013), who attributes overcrowding in accelerated projects to price markups.

Reducing the road closure time without deviating the contract duration (rows 4 and 5 in Table 3.5) allows insights into how contractors will react to having more flexibility built into contracts while still maintaining a shorter road closure duration, which is one of the main goals of the DOT. All contractors expressed the benefits of having more time to mobilize their crews and schedule work with the added time, but the case study demonstrates that the costs for this added flexibility are hardly uniform. The differences between Contractors A, B, and C can most likely be attributed to the drastic increase in risk for company B, as well as

added resource and equipment costs. Allocating four crews to one project would essentially be placing all of Contractor B's 'eggs in one basket', as the expression goes. The strong hesitancy and steep price associated with the shortened timeframe for the smaller contractor gives evidence of that fact. At the 60 day mark (the minimum time frame specified for Contractor C), Contractor C would be allocating about half of their crews to the project, resulting in less risk from a crew perspective. However, the mobilization and acquisition of appropriate resources, equipment, and expertise would be the main cost drivers. In addition to this, they expressed added risk because the project was outside the scope of their normal work, and as such would need to account for this by increasing the cost.

There are several key points that bear summarizing from the case study. First of all, the study shows that acceleration can be achieved on traditional bridge projects, the likes of which have been minimally tested in Iowa. Secondly, the cost associated with that acceleration seems to largely be linked to the size and expertise of the construction firm itself, thus amplifying the complexities of estimating acceleration costs from the outset. Larger firms will be more willing and able to manage and complete accelerated construction projects due to ample resources and crews. Conversely, with high additional risk and less resources than their counterparts, smaller firms will be less willing to bid on accelerated projects. If they do happen to bid on accelerated projects, the results of the interview and case study demonstrate that it is likely the cost will be much greater than a larger firm. When choosing to accelerate projects, contracting agencies must be aware of the potential for eliminating bidders and competition. The current Iowa DOT letting guidelines speak to this fact, but it may be all the more important when considering accelerated projects. Thirdly, reducing the road closure time and maintaining a longer, overall contract duration on

accelerated projects seems to be one possible solution to keeping costs low and ensuring bidders are not eliminated. As noted by Contractor B, letting contracts in this manner would be introducing intermediate milestones and complexity to the project. But, when acceleration is necessary the overall cost could be minimized and the schedule less complex than when decreasing both contract and road closure times.

### **Conclusions**

In order to gain a better understanding for the costs connected with accelerating traditional projects, a thorough literature review, analysis of historical bridge data, and personal interviews were conducted with three unique contractors in Iowa as well as DOT personnel. The following conclusions were drawn based upon the above summarized work:

- While flexibility is not specifically built into current contracts, significant acceleration is possible on construction projects using traditional methods. Contract duration reductions may be as much as 50-75% of the original contract length without converting construction to commonly available ABC techniques. However, a potential risk identified with increased acceleration is possible alienation of contractors, particularly those of the small-medium size. Projects with extreme compression will most likely be relegated to a select few large construction firms, which may conflict with DOT letting philosophy of encouraging competition.
- The minimum completion timeframe for bridge projects utilizing compressed traditional construction methods appears to be between 20-40 days, well under most of the current contract durations. Further reductions in time beyond this will necessitate the use of ABC techniques. Where possible, it may make sense to build

the new foundation outside of the footprint of the existing bridge before closing the road and demolishing the bridge to further cut down on closure times.

- The cost associated with this acceleration is driven largely by contractor ability and project considerations, and as such overarching trends are difficult to ascertain. Costs can increase from 10-100% + on accelerated projects, reflecting the varying level of risk assumed by different contractors.
- The impact of inspector quality and risk may overshadow acceleration capability on a project, both in terms of cost and schedule.

### **Recommendations Going Forward**

Based upon the above presented information and the developed conclusions, the following recommendations about estimating/keeping costs and schedule low on accelerated projects have been developed:

- When possible, let contracts to maximize the time between the letting date and the late start date, which will allow contractors to schedule their crews efficiently, give time for ROW acquisition and utility work to be completed, and ensure sufficient time to order materials (adequate lead times). All of these lead to a decrease in risk from the contractor perspective.
- Consider VE proposals that give more control to prime contractor by requiring the use of fewer subcontractors.

- Maintain longer contract durations but introduce critical closure periods for projects where user impact is high. This approach maintains high overall project float and gives the contractor more flexibility on projects, which should be reflected in a lower cost
- Estimate contract durations using updated project data/more quantitative methods
- Utilize A + B bidding and incentivize the user costs to a greater level
- To reify the above conclusions and recommendations let a project with a range of contract durations for which costs for all contract durations would be proposed (See Appendix I for example)

## CHAPTER 4. CONCLUSIONS

Innovations in bridge design, construction, and management are necessary in light of the depreciating state of public infrastructure and the absence of adequate funding. With the presence of more complex and/or irregular designs that incorporate long spans and tall piers, the pile-to-pile cap connection under uplift forces should be an area of emphasis for agencies to consider. For many years, the industry and many agency officials (specifically in the Midwestern region of the U.S.) have either neglected the design of this connection, or have loosely based it on two previous works from 1972 (Horn et al.) and 1982 (GAI Consultants). Regardless, the NCSC survey along with further investigation into other agency's design practice in this area revealed that many still incorporate shallow embedment-type details, the likes of which perform the same as bare piles embedded 12", or marginally worse. This is supported both by past research (GAI 1982, Heristchian 2014a) and the results provided in this thesis involving threaded rods, cage, cage with bar specimens. Three anchor configurations along with a rational design procedure have been presented in this thesis so that agencies can readily adopt the recommendations and/or use the findings to investigate further details. Ultimately this work will lead to the safe design and transfer uplift loads through the pile-to-pile cap connection. Findings also revealed the inaccuracies of designing the connection based purely on bond strength, as other factors appear to impact the performance and capacity of the connection. There is a need for further numerical and experimental research in order to properly understand the failure mechanisms that were observed and extrapolate these results onto connections incorporating details outside of the ones tested.

In Chapter 3 a larger, high level overview of projects was investigated and analyzed in order to determine the feasibility of accelerating bridge projects utilizing traditional techniques and the associated costs with this compression activity. Maximizing the efficiency in schedule and cost allocation is therefore the topic of investigation for the second half of this work. To do this historical data from Iowa was reviewed and analyzed on 285 bridge projects from 2012-2016, which provided a broad base from which future research can build on to continue developing the relationship between cost and time for bridge projects. In addition to this, contractor meetings revealed the extent to which acceleration could be achieved, the factors affecting it, and general costs that were tied with shortening the project lifecycle. Recommendations were given to continue exploring the relationship between cost and time for bridge projects, as well as options for agencies minimize costs when acceleration on projects is desired.

While there is more research that needs to be performed (as is most often the case), many practical and important steps have been gained by the research and findings presented herein. Together, the topics investigated will contribute not only to safer design practice, but also paves the way for agencies to effectively manage and balance their time and cost resources to better serve the public on bridge projects.



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## APPENDIX. SAMPLE INSTRUCTIONS FOR VARIABLE CONTRACT DURATION LETTING

The following are sample instructions that could be included in a future letting to continue exploring the relationship between acceleration and cost. The letting instructions would ask bidders to provide costs associated with multiple contract durations, as specified per Recommendation #6. The language can be altered in order to abide by individual agency policy and formatting, but should clearly communicate the intent of receiving bid prices associated with the varying contract duration periods. The example given is for the case study bridge project, number IMX-035-4(188)110--02-85, and assumes a 120-day maximum contract length (as specified in the original letting documents).

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This contract is let under special provisions titled, “Variable Contract Duration.” The purpose of this contracting method is to gather a range of bids from contractors based on a number of contract duration periods specified in the document. Different combinations of ‘TOTAL CONTRACT DURATION’ and ‘CRITICAL ROAD CLOSURE DURATION’ are included. Please read carefully and complete all of the materials herein for consideration of the award bid.

To summarize, there are two steps necessary in order to finalize the bid package:

1. Complete the enclosed bid tabulations based on the indicated maximum contract duration shown below
2. Determine the appropriate compensation required (price) for the additional contract duration combinations that are specified

MAXIMUM CONTRACT DURATION: **120** DAYS \*\*insert contract duration based on production charts – i.e. the normal contract duration\*\*

BID TABULATIONS: \*\*include the bid tabulation sheets\*\*

TOTAL NORMAL COST: \_\_\_\_ \*\*based on maximum contract duration and bid tabulations\*\*

ADDITIONAL BIDS:

	CONTRACT DURATION (Days)	CRITICAL ROAD CLOSURE DURATION (Days)	COST (\$)
<b>Critical Closures <u>Not</u> Specified</b>	90	90	_____
	60	60	_____
	30	30	_____
<b>Critical Closures Specified</b>	120	90	_____
	120	60	_____
	120	30	_____