Evaluation of the Effectiveness of the Strategic Initiative 9 Pilot Bridge Concepts

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Prepared in Cooperation with Ohio Department of Transportation
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Abstract

Ohio DOT created Strategic Initiative 9 to study ways to build bridges faster, smarter and better. This report examines the construction of 8 bridges. For three bridges, one continuous for live load stringer, one simple span steel beam with precast deck panels and an adjacent precast slab panel bridge the speed of construction was studied. For three adjacent box beam bridges, the use of lateral post-tensioning as a means of improving performance was studied. The main conclusion of the study was that the success or failure of the project was not caused by technical issues, but by human issues. Specifically, successful projects had well-drawn plans and specifications, used pre-bid and pre-construction meetings to get information to the contractor, used partnering, had all parties aligned to the same goals, had good change management and allowed significant latitude for field personnel to make decisions. It was also found that unless incentives/incentives were very large, they did not make a difference. Finally, lateral post-tensioning was found to be effective at improving the structural behavior the adjacent box girder bridge, even at post-tensioning levels less than required by the AASHTO LRFD Specifications.
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**DISCLAIMER**

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Ohio Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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CHAPTER 1
INTRODUCTION

1.1 INTRODUCTION

The State of Ohio has fourth largest highway systems in United States. Many parts of the system are old, having been built in the 1950s and 1960s, and are in need of extensive repair and rehabilitation. Increased traffic volumes necessitate widening of the roadways in some areas. To complete the needed work on the highways, road crews often must close or restrict lanes. Such closures and restrictions have real costs to motorists in terms of lost time, higher fuel costs and lost revenues. These traffic delays also make drivers angry. As a public agency, the Ohio Department of Transportation (ODOT) has an obligation to do what it can to limit the inconvenience it places on the public.

Bridges present a particular problem. When repairing pavements, it is frequently possible to use a median or other part of the right-of-way to create a temporary, additional lane. This is not usually possible with bridges. In construction sequences, bridges tend to be critical path items and any delay in completing bridge construction will usually delay the entire project.

It may appear that the solution to the problem is to find ways to speed up bridge construction, but the actual goal is not speed up the entire construction process, but rather to limit the time the bridge is either closed or under lane restrictions. ODOT created Strategic Initiative No. 9 in an effort to find ways to limit bridge “down time”. A study was conducted to identify possible solutions (Miller 2005). The study consisted of a literature and internet search for possible solutions along with a survey of contractors. Using the results of these studies, several concepts for speeding up bridge construction were identified:

1) Stay-in-place forms, either steel or concrete;
2) High performance concrete materials with reduced curing times;
3) Prefabricated bridge decks – either precast concrete or concrete filled steel grids;
4) Prefabricated superstructure units, such as steel beams with a prefabricated deck attached;
5) “Deckless” bridges, such as adjacent precast box or bulb “T” beams.

It was also found that bridge sub-structure elements could be prefabricated, reducing construction times when a complete bridge replacement or bridge widening is being done. After considerable discussion, the Strategic Initiative No. 9 Committee identified six specific concepts which would be suggested to the Districts for use in demonstration projects. The concepts were:
1) Stay-in-place steel forms;
2) Prestressed concrete bridge decks which are match cast, post-tensioned and ground to final profile;
3) Concrete filled steel grid decks;
4) Precast sub structure units;
5) High performance concrete materials which have shorter curing times;
6) Transversely post-tensioned adjacent box beams with integral wearing surfaces ground to final profile and compatible with standard TST-1-99 railings. Precast parapets are to be considered when over the side drainage is not acceptable.

It has been suggested that the concepts listed above would be more effective at limiting the time bridges are closed or under lane restriction if they were combined with innovative contracting techniques. Ohio DOT has identified some promising contracting techniques:

1) Work day contracting - sets the number of construction days to complete a project or portion of a project.
2) Incentive/disincentive - contractors are paid extra for projects or tasks completed ahead of schedule and are assessed deductions for projects or tasks completed behind schedule.
3) Lump-sum incentive - large lump sum payments are used as incentive to meet a specific completion date.
4) Liquidated Savings - contractors are paid an amount to late finish liquidated damages.
5) Design/Build - allows the contractor to create a design which will optimize the construction schedule.
6) A+B - A is the cost of the project, B is the contractors estimate of project completion time. Bids are judged on lowest total "cost" using A + B*user cost/day. B becomes the contract completion time with penalties for late completion.

1.2 PURPOSE OF THIS PROJECT

This project sets out to examine the construction of six different bridges. The bridges are:

1) GUE-513-1.80 This is a prefabricated, prestressed slab bridge located in Guernsey County. The project was an accelerated construction project with a goal of constructing the bridge in 16 days. Only the superstructure was replaced.
2) PIC-22-17.03 This is a continuous for live load steel structure located in Pickaway County. The project was a design/build accelerated construction project with a goal of constructing the bridge in 60 days. Only the existing foundations could be reused.
3) CLI-730-11.13 This is a longitudinally pretensioned, laterally post-tensioned, adjacent box girder bridge located in Clinton County. This project had two
objectives. The first was to try lateral post-tensioning. Although common in some states, it was rarely used in Ohio prior to the construction of CLI-730. The problem with adjacent box girder bridges is that the shear keys crack and leak. It is believed that lateral post-tensioning will compress the keys and prevent cracking. The AASHTO LRFD (2007) specifications also give laterally post-tensioned box girder bridges a more favorable distribution factor. The second objective was to see if the construction time could be shortened by casting an integral wearing surface on top of the boxes rather than casting a composite concrete surface or applying an asphalt layer.

4) MOT-70-14.74 This is a longitudinally pretensioned, laterally post-tensioned, adjacent box girder bridge located in Montgomery County near the Dayton Airport. The project was repeat of CLI-730.

5) FAI-22-15.25. This is a longitudinally pretensioned, laterally post-tensioned, adjacent box girder bridge located in Fairfield County. This was to provide another data point for laterally post-tensioned box girder bridges.

6) HAN-75-15.99 This is a single span, steel stringer bridge with post-tensioned, precast deck panels. The purpose of this project was to see if construction time could be shortened by using precast panels.

In each case, the project either tried to shorten construction time or improve the quality of the bridge, or both. It should be noted that “shortening the construction time” refers only to the actual time the bridge is closed or has a traffic restriction. Thus, a project would be considered a “success” if the time of bridge closure or restriction is reduced, even if the overall project length is increased.

Author’s note: For readers unfamiliar with the ODOT numbering system, all bridges are numbered using the sequence aaa-xxx-yy.yy. The first 3 letters indicate the county. The numbers after the first dash are the route number. The bridge may be on the road or, often in the case of an interstate, go over the road. The final numbers indicate the miles from either the west or south border of the county. Thus, GUE-513-1.80 is located in Guernsey County (east of Columbus and Zanesville), on State Route 513, 1.80 miles from the southern border of the county.
CHAPTER 2
A METHOD FOR BENCHMARKING

2.1 INTRODUCTION

One of the goals of the research is to determine if various methods of bridge construction result in bridges built “Smarter, faster and better.” In order to find any improvement in bridge construction techniques, it is first necessary to benchmark the performance of current practices. The problem is that every bridge is, in some ways, unique. Thus, trying to compare two bridges can be a difficult task.

Another difficulty is that new data is constantly being added to the set. Costs change constantly. Bridge design methods change as do construction methods. Thus, benchmarking data has a limited shelf life. The purpose of this chapter is to suggest and validate benchmarking method for bridges. Much of the data in this study focuses on the adjacent box girder bridge. Therefore, the adjacent box girder bridge will be used as the basis for developing the benchmarking method.

The work in this chapter is a condensed version of a Master’s Degree Thesis (Stone, 2007). More complete details can be found in the thesis.

2.2 BENCHMARKING DATA

2.1.1 General

The data used for this study came from ODOT construction records of adjacent box girder bridge replacements built between 2002 and 2005. A total of 54 projects were identified. However, some of these projects were to construct multiple bridges and some others involved extensive amounts of work other than the bridge replacement. In these projects, it was almost impossible to separate out the data specific to the bridge, so these projects were eliminated. Projects with incomplete data were also eliminated. This left 30 projects.

The intent of Strategic Initiative 9 is to find ways to build bridges faster and/or with better quality. It is not to necessarily build the cheaper, but cost is certainly a factor. In the end, any improvement construction time and/or bridge quality must be judged against any increase in cost. Of course, if a bridge can be built faster, better AND less expensive; that would be ideal.

To provide a benchmark for the stated goals, it would be necessary to measure:

1) Basic parameters, such as number of spans, span width, whether the construction was phased or detoured, and any special features.
2) The amount of time the bridge was closed or under restriction. The goal is not necessarily to reduce overall construction time, but to reduce the time the bridge is closed or restricted.
3) The cost of the bridge.
4) Measures of bridge quality.

Quality of the bridge is a long term measurement. It may take several years before any distress from poor quality materials and/or construction is evident. Thus, measurements of quality are not included in this study. Only items 1-3 were evaluated.

Data for the bridges was collected from project plans, ODOT Construction Management System (CMS), and contractor provided progress schedules. CMS is a computerized record keeping system in which ODOT keeps information regarding finances, key dates, items of work, and change orders for each project. Contractors provide progress schedules to ODOT, which list the sequence of activities that will be performed and estimated time to complete each activity.

Gathering data about the parameters for each bridge was quite easy as this data was readily available from the plans. Information about cost and construction time was more difficult.

2.2.2 Cost Determination

Determining the bridge cost is not straightforward. Overall project cost cannot be used because each project contains cost items which are not related to the bridge construction. These items are not uniform between projects.

ODOT does require the contractor to break out individual costs. There is an area of the CMS where the contractor is required to break out the costs of the structure and this line item was used to determine the bridge cost. However, using this number has some problems:

1) The “structure cost” is determined by those items listed on the plans as structural items. There does not appear a uniform method of determining what is put into this line. For example, in some projects, the approach slab is listed as part of the bridge, in others it is part of the roadway paving.
2) There are differences in the bridges which affect the costs. Does the bridge have a sidewalk? Does it use guardrail or cast-in-place concrete barriers? Is the construction phased?
3) The particular situation of the contractor affects the costs. ODOT awards jobs based on total cost. It is possible for two bids to have nearly identical total cost, yet have widely variant costs for individual items. For example, assume two bidders obtain quotes for asphalt and concrete. They use different suppliers. One may pay more for concrete and less for asphalt, while the other pays less for concrete and more for asphalt. The structure cost would be higher if the first bidder wins.

Unfortunately, there is no easy way to correct for these items. Thus, for this study, the items marked as “structural” were used as the bridge cost without modification.
2.2.3 Time Determination

Ideally, the time to build the bridge should be determined as the day the bridge was closed or placed under restriction to the day the bridge was fully opened to traffic. For a more accurate comparison, weather days (days when no work is done due to bad weather) and time lost to unusual delays (strikes, material supply interruptions, etc.) should be subtracted.

There are two possible sources for this information. The first are the daily construction diaries kept by ODOT personnel. The other is the contractor’s progress schedule. However, neither of these proved to be a reliable source of information.

The construction diaries vary widely in the information provided. All provide the overall project start and end date. Most (90%) indicate when the bridge/road was opened to traffic. Only 7% listed the road closure date. There was a wide variation in detail. Some listed when every significant item was completed and accounted for delays. Others had almost no detail. Every project listed the number of weather days granted, but the actual days missed were often not known, so it was not known if these weather days occurred while the bridge was closed or not.

Contractor progress schedules are not much better. They vary from hand drawn charts to computer generated CPM schedules. They are not done in any consistent way. Some contractors spell out every procedure (e.g. form deck, place bar, pour deck, cure deck) while others group procedures (e.g. the item is listed as cast and cure deck, but implicitly includes forming and placing rebar). These schedules are created at the beginning of the project and may or may not be updated.

For this study, the construction time was taken as road closure date to road open to traffic date. If the road closure date was not known, the project start date was used and the project end date was used if the road opening date was unknown. Because delay information was often missing or was kept in an inconsistent manner, delays were not subtracted from the project duration.

2.3 BRIDGE DATA

The data shown is for 30 adjacent box girder bridges built between 2002 and 2005 where sufficient data was available for analysis.

Table 2.1 Bridge Data

<table>
<thead>
<tr>
<th>Number of Spans</th>
<th>Number of Bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
</tr>
</tbody>
</table>
For analysis purposes, the bridges were grouped as single span or multspan.

Figure 2.1 shows the structure cost. For a more accurate comparison, Figure 2.2 shows the structural cost per square foot of deck area, as this is a common figure used to compare bridge costs. Figure 2.3 breaks this down by single span vs. multspan.

Figure 2.1 – Structure cost.
Figure 2.2 – Cost per square foot of deck.

Figure 2.3 – Cost per square foot broken down by structure type.
The data show that in the period examined, $119 per square foot was the average cost, but Figure 2.1 shows there were 3 very expensive bridges. In this case, the median cost of $101 per square foot is probably more accurate. It was thought that multispan bridges would be more expensive due to the need to construct piers, but this was not the case. Multispan bridges are actually 10% cheaper. The data also show that for 87% of the bridges studied, the structural cost was 60-80% of the overall project cost.

Figure 2.4 shows the time of construction. While the average shows 144 days, there are clearly two outliers in this data. The median of 120 day is probably a more reasonable value. There was no difference between single and multi-span bridges. However, the data tell much more about the time of construction. According to the data, the 25th percentile value as 98 days, meaning that only 25% of the bridge were finished in less 98 days. The 75th percentile was 185 days. Thus, 50% of the bridges take between 3 and 6 months to finish. This is most, if not all, of the construction season.

There was no correlation between the cost of the bridge and time of completion. More expensive bridges do not take longer to build nor does it cost more to build bridges faster.
An analysis was done of the bridge characteristics to see if some types of bridges simply take longer to build and/or are more expensive than others. For this analysis, bridges were broken down into four categories:

1) Single span – bridge closed to traffic during construction (12)
2) Single span – phased construction (7)
3) Multiple span – bridge closed to traffic during construction (10)
4) Multiple span – phased construction (1)

Since there was only one, multiple span, phased project; this type of bridge was dropped from the analysis. Tables 2.2 and 2.3 show cost and time of completion for the various types of bridges. For Table 2.2, one of the non-phased, single span bridges, had a cost of over $400/sq. ft. This was eliminated as an outlier. Similarly, the two multiple span bridge that took over 300 days to build were eliminated from Table 2.2. The data clearly show that phasing the construction increases construction time and cost.

Table 2.2 – Cost per square foot

<table>
<thead>
<tr>
<th>Type of Bridge</th>
<th>Average</th>
<th>Median</th>
<th>STD</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Span, non-phased</td>
<td>$114</td>
<td>$106</td>
<td>$45</td>
<td>0.40</td>
</tr>
<tr>
<td>Single Span, phased</td>
<td>$124</td>
<td>$105</td>
<td>$47</td>
<td>0.38</td>
</tr>
<tr>
<td>Multiple Span, non-phased</td>
<td>$93</td>
<td>$84</td>
<td>$28</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 2.3 – Days to completion

<table>
<thead>
<tr>
<th>Type of Bridge</th>
<th>Average</th>
<th>Median</th>
<th>STD</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Span, non-phased</td>
<td>108</td>
<td>79</td>
<td>78</td>
<td>0.72</td>
</tr>
<tr>
<td>Single Span, phased</td>
<td>170</td>
<td>184</td>
<td>34</td>
<td>0.20</td>
</tr>
<tr>
<td>Multiple Span, non-phased</td>
<td>114</td>
<td>114</td>
<td>32</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Oddly, multiple spans bridges do not take appreciably longer to build than single span bridges. Although multiple span bridges cost more in absolute cost, on a per square foot basis, they are actually cheaper than single spans. This is because the fixed costs are spread out over a larger structure and it is less expensive to build piers than abutments.

An analysis was done to determine if the feature the bridge spans has any impact on time of completion or cost. For example, it was thought that bridges over major roads would be more expensive or take longer to build due to accessibility problems or that bridges over large rivers would be more expensive or take longer to build. In general, there was no relationship between the size of bridge and what it crossed and the time of completion or cost.
2.4 BENCHMARKING ANALYSIS

One objective of benchmarking is to look at projects which perform well and determine if there were some characteristics of the project which contributed to the better performance. Similarly, poor performers may provide “lessons learned”, which would improve subsequent performance. An analysis was done of some of the better and poorer performers based on information provided in ODOT records. Unfortunately, this analysis was limited by the lack of consistent information in these records. For brevity, only a summary is presented here. The detailed analysis can be found in Stone (2007).

The overall conclusion will be a theme throughout this report. “Building Bridges Smarter, Faster and Better” is not a technical problem. The technology to build bridges quickly and efficiently exists. The factor with the largest influence is the human factor.

Two the bridges judged “best performers” in that they were constructed in a short amount of time and at low cost, were built by the same contractor. Data from the construction diaries show the contractor specialized in structures and had devised an efficient system for construction.

The bridges judged worst performers, in that they had long times to completion and were among the highest cost, all had non-technical problems. In the case of the bridge with the longest time to completion and one of the highest costs, the problems were:

1) There was a dispute over whose responsibility it was to obtain a required permit.
2) There were problems with equipment being inadequate and/or breaking down.
3) There was a labor problem.
4) There was a dispute over the adequacy of pile depth.
5) There was an intentional delay in opening the road so the contractor could complete other work for the city.
6) The project was stopped due to winter.

It should be noted that EVERY project studied has some type of problem; weather, flood, equipment, materials, etc. One difference between the better performers and worse performers was the ability of the contractor to handle these delays. Another big difference was the “partnership” between ODOT and the contractor. This point will be discussed in detail in Chapter 8, Conclusions and Recommendations.

Finally, financial incentives and penalties do not seem to make much of a difference. There were bridges finished on time or early and at low cost with little or no incentive payment. It was found that time extensions were almost always granted and penalties were often waived or reduced. Again, this item will be discussed in more detail in Chapter 8.
2.5 RECOMMENDATIONS

1) ODOT should establish a “benchmarking team” to track benchmark data for bridges. This team should be charged with establishing the data needed and determining how this data should be collected.

2) There needs to be a standard for project cost data. As previously noted, a given item might appear in different cost areas in different projects. Creating a consistent definition of where the cost of a specific item appears would allow more accurate tracking of cost data.

3) Create a form for the construction diary so that consistent data is obtained. The diaries should clearly state milestone days – date the project starts, date the road is closed, dates bridge construction begins and ends, date of road opening and final project date. These milestones should be adapted, as needed, for phased work.

4) Be sure the construction diaries clearly track and account for delays. Delays dates should be included so that the impact on a specific construction task is known.
3.1 DESCRIPTION OF THE PROJECT

The first bridge chosen for the SI-9 program was GUE-513-1.80 located in the town of Quaker City, Ohio (approximately 50 miles east of Columbus). The existing bridge was a 2 span, concrete slab bridge, with each span being approximately 30 foot long. The superstructure was to be replaced, but the existing pier and abutments were to be re-used after repair. Another slab bridge was the logical replacement structure.

This bridge was chosen for rapid replacement. Because the site is in a rural area, the detour was approximately 25 miles for cars and 40 miles for trucks and large busses. Local officials were concerned about detouring school busses over this long distance. They wanted the bridge built after the school year ended and with as little interference with the summer school term as possible. Another major factor in the decision to accelerate the construction was a festival held in Quaker City each summer. The festival is a major revenue source for the local community and it was desirable to have the bridge completed before the festival.

The engineer decided to use a post-tensioned, adjacent concrete slab unit for the superstructure. The rail would be precast concrete as well. Figures 3.1 –3.6 show the details of the bridge structure. It was recognized that accelerating construction was not simply a matter of using precast elements which could be assembled quickly. Careful planning and changes in “normal procedures” were also needed.

3.2 PRECONSTRUCTION MEETINGS

ODOT normally holds preconstruction meetings on all projects. For this project, ODOT and the contractor used the meeting to discuss potential problems before construction started. To assure there was enough time to resolve any potential problems, the meeting was held 6 months before construction was to begin.
Figure 3.2 - Elevation of the Replacement Structure

Figure 3.2 - Longitudinal Sections of the Deck. Single Panel (above) and Entire Length (below)

Figure 3.3 - Cross Section of the Deck
Figure 3.4 – Plan View

Figure 3.5 - Typical Panel
Bridge construction was an accelerated process to be completed in 16 days with work being scheduled between June 16th and June 30th, 2003. The proposed work included the following:

1. Removal of portions of existing concrete deck, sidewalk, railing and substructure units
2. Construction of new portions of a cast-in-place abutment and wingwall;
3. Setting of the precast bridge units, precast approach slab and precast railing;
4. Grouting of the beam seat areas and the areas below the approach seats;
5. Grouting the shear keys;
6. Lateral and longitudinal post-tensioning;
7. Grouting post-tensioning tendon ducts;
8. Grinding and grooving the final riding surface;
9. Placement of the sidewalk concrete
10. Epoxy coating the sides of the fascia girders and general clean-up.

The meeting was attended by representatives from ODOT, the general contractor, the post tensioning subcontractor, the precast fabricator and the design engineering firm. Among the issues discussed were:

1. The contractor intended to employ two work shifts of eight hours each and work 7 days each week. Quaker City has a noise ordinance which would preclude this. Since this was to be a short duration project, ODOT indicated
that it would approach local officials and neighbors affected by the noise and secure permission to work the needed hours. One unusual condition was noted. There was a funeral home next to the bridge and it would be necessary to curtail noise if there was a funeral being conducted. Since this was an unlikely event (given the size of the town) and since it was also unpredictable, the decision was made to use the proposed schedule and adjust it as needed. As it turned out, the funeral home was not an issue during construction.

2. The contractor was concerned with the limitation on how long he could close the bridge. To meet the schedule, he asked to be able to saw cut existing slab from the abutments before actually closing the bridge. ODOT allowed this after the contractor had an engineer investigate the issues related to structural stability and safety.

3. The plans required the contractor to conduct a mock up test fit on the modular slab units in the precasting yard (Figure 3.6). This was to assure the correct alignment of the ducts for the post-tensioning strands. The mock up test would also help in checking the basic fit of the beams as any problems which occurred once the installation began at the site would be difficult to resolve within restricted time frame.

4. It was decided that weather days would be allowed as the contractor cannot control the weather. ODOT rules permit that a weather day is allowed only if less than two hours of work is performed on that day and this turned out to be a critical issue. The two hour rule makes sense with respect to an 8 hour day. But what if the contractor can only do 3 hours work on a scheduled 16 hour day? This was determined to be a negotiable issue.

5. During both the pre-construction meeting and the mock up assembly, the precaster expressed a concern about the post-tensioning. The normal procedure is to post-tension one or two strands, measuring force and elongation. This data is then sent to the ODOT engineers in Columbus for verification and approval, which often takes a few days. Such a delay would be unacceptable in an accelerated project. ODOT agreed to have engineers on site to verify the post-tensioning.

3.3 MOCK-UP TESTS

Special provisions in the contract provided for a full mock up test for the bridge assembly in the yard before the final assembly on the site (Figure 3.6). This requirement was included to check for proper fit-up and alignment, and to verify that every beam unit was constructed in compliance with all plan requirements. The contractor was to use blocking to stimulate the beam seats elevations at the abutments and pier. Unfortunately, no one remembered that the bridge had a slope and the blocking was not sloped. This became an important issue during construction as misalignments were found when the bridge was assembled on the actual abutments. Post-tensioning cables were installed in all the slabs (but not tensioned) to verify duct alignment.
3.4 CONSTRUCTION

The construction work began as scheduled on June 16th, 2003 and was completed on July 3rd. This was 3 days behind the original plan of completing work by June 30th. The significant delays were:

1. The contractor had intended to work two 8 hours shifts per day. The contractor actually worked a single shift each day which averaged about 12 hours per day. This has the effect of losing 4 hours per day of work and increasing labor costs as the contractor must pay time and a half for the extra hours. The contractor cited two reasons for this. There was dead time in the day (e.g. waiting for grout to cure) and there wasn’t justification for the additional shift. It was better to employ a single shift for however many hours were needed each day. The contractor also thought the overlap time between shifts would be too long. The supervisors would have to explain, in detail, every aspect of the work to the incoming supervisor. The incoming supervisor would then have to repeat the process with the workers. This is a time-consuming and tedious process. For each shift there is always some start-up and finish-up time (e.g. getting out or putting away personal tools). The contractor determined that a single shift would be most efficient.

2. Some of the delay was due to rain, which occurred on 3 different days. This pointed out a problem with the current ODOT rules. Current rules allow a weather day only if the contractor works less than 2 hours. This presents some incentive to the contractor not to work if he cannot work the entire day. Since the contractor was working extended shifts, the number of hours lost could be significant. In a fast track situation, it is to ODOT's advantage to have the contractor work as many days as possible, even if it is only partial days. Fortunately, ODOT used a partnering system. When a situation like this arose, the contractor and ODOT representatives met to find a mutually beneficial solution. As a result, ODOT granted some relief for weather even if the contractor worked more than 2 hours in a day.

3. Some of the delay was due to equipment problems. On the first two days the track hoe used to break up the old concrete broke down. On the 10th day, there were problems with a crane. These delays could have been avoided by reserve equipment available, but it is not clear if this would have been cost effective. An essential part of the fast track process is a cost-benefit analysis of the cost of reserve equipment and probability that reserve equipment will be needed.

4. The deck on the old structure was removed by using a rock drill, which was a time consuming process. It was suggested by the site supervisor that saw cutting the deck and then removing it with a crane might have saved time. Here pre-project planning would have helped.

5. On day 5, the wrong deck units were brought to site. This caused a delay until the correct units were shipped. This would not have been much of a factor on a normal job, but it was important on an accelerated project and shows the need to verify all shipments in these cases.
6. Field verification problems also caused delays. The plans of the bridge did not mention in detail the exact amount concrete which needed to be removed from the wing walls. The contractor also neglected to mark the centerline of the existing bridge before removal. This was a problem because the new deck was not centered on the re-used pier and there was some confusion about the placement of the new deck. A surveyor was brought to the site to determine the centerline, but this caused a delay. It is always important to verify site conditions before construction, but for an accelerated job it is even more critical.

7. As noted in the section about the mock-up, a problem occurred during the placement of the beams. The mock up test was carried out on flat ground, but the abutments were sloped and this was not accounted for in the mock-up. Significant adjustments were needed for the correct alignment of the beams on site, requiring some “field engineering” of the shims. In the end, wood shims were used, which ODOT normally would not allow. This points outs a significant risk in accelerated construction. When problems occur, the owner must often choose between delays or less than optimal solutions to problems. The alignment problem also occurred with the PT ducts. The ducts were aligned perfectly during the mock up, but on site it took some effort and time to get them in line.

8. The grout was a problem. The type of grout needed for the shear keys (between the girders) was specified in the contract as a non-shrink grout with a minimum strength of 6000 psi prior to transverse post tensioning. Exposed concrete surfaces at blockouts and recesses had to be treated with a bonding agent prior to filling. However, it appears that parts of these provisions were overlooked by the contractor in the initial bid phase. The contractor then had to work quickly to find a suitable grout. The grout initially chosen by the contractor was not on ODOT’s list of accepted grouts and so it was rejected. A second grout was chosen from the approved list. The grout took almost twenty hours longer to set as compared to the one chosen by the contractor (this contributed to the contractor’s decision to use one shift). The grouting company was not familiar with the type of grout used and had difficulty working effectively with the grout. The grout did not come up to the required strength, although there were questions about whether the grout testing was done correctly. Normally, the grout between the beams is not an issue. The inspector verifies that the grout is mixed to the manufacturer’s specifications and the grout is then poured into the keyway. On a typical ODOT design, the grout is covered by waterproofing and asphalt or a cast-in-place composite deck. Since the grout is covered, no additional testing is done. For this bridge, the grout joints are exposed so a tighter quality control was needed. Testing procedures for the grout were available, but since the grout is usually not tested, no one thought to bring the testing equipment. Make-shift cylinders made of plastic pipe were used for specimens. In addition to the testing problems, the grout shrunk after curing and the cracks in the joints had to be sealed with high molecular weight Methacrylate (HMWM).
9. The bridge was designed to use the top surface of the beams as the final riding surface. After placement, post-tensioning and grouting, the tops of the beams would be ground to profile and grooved. However, the lifting lugs and post-tensioning grout tubes protruded from the top flanges of the beams. No one thought to leave dish-outs around these protruding elements so that the contractor could cut them back below the final riding surface. As a result, the contractor had to create these dish-out by jack-hammering them into the top flange. The protruding elements were cut back and the dish-out was covered with grout.

The contractor completed work such as site clean-up, sealing the sides of the beams, etc., after the bridge was opened. This is an exception to the normal A+B contract and was made to allow the bridge to open. This exception was in the spirit of the accelerated construction philosophy. The key was to get the bridge open as soon as possible, even if the overall project time was not shorter.

3.5 POST CONSTRUCTION OBSERVATIONS

3.5.1 Post Construction Meeting

An essential part of the SI-9 project was to hold a post-construction meeting to determine what worked and what needed to be improved. This meeting was attended by representatives of the ODOT, the prime contractor, various sub-contractors, the design engineer and the research agencies.

The first issue raised was the grout. The engineer specified a fast-set grout without determining specific products which could be used. He later stated that he assumed the contractor would do some research on the grouts available. Unfortunately, unusual grout specification was not clearly flagged in the contract documents and contractor missed it. This created a problem as the contractor did not account for the unusual grout in the bid and there were delays because the contractor did not have an acceptable grout when the project started.

The contractor wanted to use a quick setting grout which was not on ODOT’s approved list. ODOT denied the contractor’s request and asked the contractor to use a grout on the approved list. All of the approved grouts required at least 24 hours to set. As previously noted, waiting for the grout to set caused a delay. The Contractor suggested that ODOT should either add more materials to the approved list or completely do away with it; perhaps replacing it with a performance based specification. Giving the contractor greater flexibility may help in cutting down the construction time.

It was also noted at this meeting that a partnering relationship developed between the ODOT and the contractor. All parties noted that this was very helpful in getting work done quickly and amicably. The role of the project manager also came up. It was observed that it is usually difficult to contact or seek immediate advice/suggestions from the ODOT engineer or the design engineer. In this project, the manager assumed more of
a proactive role and provided the necessary clarifications whenever they were sought. He was also given more authority to make decisions and this helped keep the project on track.

The contractor also mentioned that in a project of short duration like this one, motivating the entire team is a difficult. Tasks cannot be spaced out, but need to performed one after the other and there is no luxury of thinking about the problems. Problems adversely affect the morale of the team.

3.5.2 Observations by the Research Team

In the end, the Quaker City Bridge Project was a success. The bridge was completed in 19 days rather than 16 days, but it was still completed within a reasonable enough time that project’s objective, having the bridge open for school bus and festival traffic, was accomplished. The research team made several observations:

1) The incentive on the project was $5,000/day, $25,000 maximum. The penalty was $5,000/day. State law limits these values. The research team noted that these are not large incentives or penalties. While these incentive/disincentives will work to speed up construction, they will do so only in a limited way as the dollar values are not that high. In another SI-9 project (PIC-22), the incentives and penalties were much higher and contractor responded by bringing in his best crews, bringing in extra crews, looking for innovative ways to speed up construction, etc.

2) Construction was slowed by equipment break-down. The research team wondered why spare equipment was not available. This requires a risk vs. reward analysis.

3) Grout selection and quality control of the grouting process are major issues. These same issues came up on several SI-9 projects. The issues regarding selection were discussed in the previous section. There is a need to have better QA/QC for the grouting process. Unlike in other ODOT projects where the grout work is usually covered with a layer of concrete or asphalt, the grout in this case remained exposed. It seemed that the need for better grout QA/QC was missed.

4) The fact that the bridge was prefabricated greatly contributed to the speed of construction and post-tensioning it made the design efficient. The post-tensioning was also an issue only because the number of contractors who do such work is limited as is the number of suppliers of post-tensioning materials. The precaster noted that he got no response from some post-tensioning material suppliers when he asked for bids. There is a concern that, in the future, the ability to construct post-tensioned bridges quickly may be limited by available contractors.

5) In a project of this nature the margin of error is very small. There are many ways in which the project could get off schedule. Faulty or broken equipment, inclement weather, shipping delays/problems and material/labor availability could all affect the schedule. The contracting agency could ask for
a contingency plan from the contractor even before the project begins. This could offer a certain degree of assurance that even if the project is delayed for unavoidable reasons the contractor is capable of getting it back on schedule.

3.6 INSPECTION OF THE BRIDGE AFTER 3.5 YEARS

In December 2007, bridge GUE 513-0180 was inspected by the research team. The inspection showed:

In general, the structure was in good condition with the following exceptions:

1) There was extensive cracking at the acute corners (Figures 3.7 and 3.8) at beams 1 and 12 (refer to 3.4 for beam numbers). This appeared to be due to some problem with the bearings. As was noted previously, there were alignment problem during final assembly which necessitated shimming the bearings. Whether the cracking was caused by the adjustments made during assembly or due to another cause is not certain.

2) The joint between beams 1 and 2 appeared to be leaking near the center pier.

3) The bridge used precast approach slabs. The rear approach slab is good condition (refer to Figure 3.4 for position) but there is extensive cracking in the forward approach slab. The forward approach slab joints were deteriorated (Figures 3.9 – 3.10)

4) There is cracking at the south wing wall (Figure 3.11).
Figure 3.9 – Cracking in the Forward Approach Slab

Figure 3.10 - Forward Approach Slab Joints

Figure 3.11 - South Wing Wall – Rear Abutment Joint
CHAPTER 4

BRIDGE PIC-22-17.03
CONTINUOUS FOR LIVE LOAD STEEL STRUCTURE

4.1 DESCRIPTION OF THE PROJECT

PIC-22 is located on State Route 22 just west of Circleville, Ohio (south of Columbus). The original structure, built in 1957, was a six span, steel girder bridge spanning the Scioto River, a major Ohio river. The replacement structure was 44 feet, 2 inches wide, supported on 5 girder lines (expanded from the original 29 feet, 4 inches wide and supported on 4 girder lines). The total structure length was 630 feet, consisting of two, 90 foot long end spans and four 112.5 foot long center spans.

PIC-22 was chosen for rapid replacement because State Route 22 is the only route across the Scioto to Circleville and long term closure or restriction would have a significant, negative effect on the local farming economy. There is also the Annual Circleville Pumpkin Show in October. This festival is a major cultural and economic event and closure or restriction of the bridge would create a major hardship for the festival, probably resulting in lost revenue. Further complicating matters, the Scioto River site was classified as environmentally sensitive, restricting the construction methods which could be used. The simple solution would have been to construct either the permanent bridge or a temporary bridge next to the original structure, but roadway alignment issues and the presence of an adjacent rail bridge eliminated this option.

4.2 SPECIAL FEATURES AND TECHNIQUES

ODOT calculated the cost of the replacement bridge at $5 million and that it could be completed in 45 days. An incentive of $50,000 per day, up to $500,000 was offered. The winning bid was $2.7 million, but with a time frame of 60 days. ODOT agreed to this, but no weather days would be granted.

PIC-22 was a design/build project. To meet the requirements of the project, the engineer and contractor used the following techniques:

1) The contractor could not go into the river due to the environmental considerations, so they elected to demolish the existing bridge in stages and use the remaining parts of the existing structure as a platform to erect the new structure. Other work was done from the flood plain.

2) The existing pier foundations were reused. To accommodate the additional girder line, a new pier was constructed (Figure 4.1). The new pier was constructed prior to closing the bridge. This did not affect the schedule as the 60 day limit applied only to bridge closure.

3) Prefabricated steel pier caps were used (Figure 4.2).
4) The structure was designed as a continuous for live load steel structure. This is a common technique for precast concrete bridges, but rare for steel. The steel girders are erected as single span girders and carry their own dead load as simple spans. Continuity is achieved by pouring a continuity diaphragm and using the slab as a reinforced concrete negative moment connection (Figure 4.3). The slab and diaphragm are poured at the same time. The beams carry the slab dead load as simple spans. Once the concrete reaches strength, all subsequent loads (future wearing surfaces, barriers, live loads) are carried as continuous spans. The engineer chose this system for speed of erection.

5) Stay-in-place steel forms were used for fast forming of the deck. To get the forms to the proper elevation and to hold them in place, steel “ladders” were used (Figure 4.4). Normally, these must be adjusted in the field. The contractor used the actual bridge girders to create a mock-up and prefabricated the ladders to save time.

6) High performance steel girders were used to increase durability.

7) Innovative bearings were designed (Figure 4.5).

4.3 CONSTRUCTION

To achieve the desired 60 schedule, ODOT had to completely close the bridge. The detour for this route was 20 miles long. This would increase the response time of the local Emergency Medical Services to an unacceptable length, so ODOT paid for the establishment of a satellite EMS unit on the other side of the bridge for the duration of construction. Electronic signs were placed on roads around Circleville urging motorists to take alternative routes to alleviate congestion on the detour. While the official detour was 20 miles long, there was an “unofficial”, and much shorter, local detour. This “unofficial” detour used a one lane bridge with a severe limit on weight. Realizing that heavy trucks might use the bridge anyway, ODOT provided for inspection of this bridge when the project was over.

Once construction started, the contractor began by working 24 hours a day. The contract required the contractor to construct a trestle next to the bridge to act as the construction platform, but the contractor did not intend to build the bridge this way. However, the contractor did construct a trestle, little more than a walkway, to comply with the contract. At this point, the contractor took on some risk. The contractor elected to demolish the existing bridge in stages, using the undemolished parts as a platform for demolition of the old structure and erection of the new one. Some of the tasks were carried out from the flood plain. Had the contractor encountered any problems (such as flooding), they would have had no recourse to ask for additional time.

After the initial demolition and erection period, the contractor cut back to one shift a day. However, subsequent weather problems and delay caused by flooding put the contractor behind schedule. A second shift was added.
Figure 4.1 – New pier being constructed before the old bridge is demolished.

Figure 4.2 – Galvanized, prefabricated steel piers and caps.
Figure 4.3 – Diaphragm and negative moment connection over the piers.

Figure 4.4 – Stay-in-place forms and ladders.

Figure 4.5 – Bearings
One of the more innovative features of the bridge was the use of prefabricated piers. The existing pier foundations could be reused, but pier had to be replaced. Also, the existing piers and foundations were not wide enough for the new bridge. Prior to closing the bridge, the contractor drove piles next to the existing foundation and constructed a pier for the extra girder line (Figure 4.1). During demolition, the contractor removed the old pier. A concrete extension and grout pad was poured on the existing foundation to level it and bring it to the correct elevation. The galvanized, prefabricated steel pier was then placed on foundations (Figure 4.2). In some cases, the contractor could not install the piers in time to prevent a delay in erecting the steel girders. Here, the contractor erected the girders onto temporary shoring and installed the piers later (Figure 4.6).

During transportation of the piers, one of the trucks was found to be slightly overweight on one axle. The driver was able to bring the truck into compliance by changing the axle spacing. However, the Highway Patrol would not accept the permit as the truck did not have the axle spacing in the permit. This caused a delay of several hours; an insignificant delay in a normal project, but a huge delay in an accelerated project.

Another transportation problem occurred during erection of the end spans. Since the girders were short, two could be shipped on a single truck but because there were an odd number of girder lines, one truck had one girder for the east end of the bridge and one for the west end. The contractor had to unload and erect the first girder and then wait for the truck to take the 20 mile detour to the other end.

As the girders were being erected, the contractor assembled the stay-in-place forms. The contractor was able to prefabricate the support ladders using a mock-up of the beams prior to starting construction, saving time. The diaphragm steel and deck steel were placed and the deck and diaphragms were cast 38 days from the start of the project.
To save time, the contractor inserted the steel for parapet walls into the wet deck concrete so that they would not have to be drilled in later. The parapet walls were slipformed. After curing, they were sealed with new sealer which had a 3 year guarantee.

Except for some delays due to weather, the project proceeded smoothly. The contractor finished 10 days early and earned the maximum $500,000 incentive. Even with the incentive, the project still cost 65% of the original $5 million estimate. Initially, ODOT had hoped to complete the bridge in 45 days, but took a bid of 60 days. The actual completion time was 50 days.

4.4 POST-CONSTRUCTION OBSERVATIONS

Without question, PIC-22 was the most successful of the projects. The success of the project can be attributed to the following:

1) Extensive pre-project planning. This is a critical point. It takes a certain amount of time to build a bridge but the secret to building a bridge quickly is to spend time on planning and other activities which can be done before the bridge is closed so that closure time is kept to a minimum. While there are many places where the effect of planning shows, the piers provide the best example.
   a. The engineer was able to reuse the existing foundation, saving time.
   b. To expand the bridge, the piers had to be expanded. The engineer did this by using a second structure which could be built before the bridge was closed.
   c. The engineer and contractor used an innovative prefabricated steel pier and cap. This required extensive planning. The bridge codes do not expressly cover this type of structure, so the design was complicated by trying to get code interpretations.
   d. The contractor and the engineer worked with the pier fabricator up front, getting input from the fabricator during the design phase.

2) The bridge was designed for fast construction. Again, the key here was spending time up front so that the time did not have to be spent during construction. The best example of this was the use of the continuous for live load design. The engineer first had to determine if this was a good design concept and if would actually save construction time. Once the decision was made to use the continuous for live load design, the engineer had to work with the bearing people as special bearings were needed. The negative moment connection had to be designed. By spending more time on design, time could be cut out of construction.

3) The incentive was large enough. The incentive in this project was $500,000 maximum. The contractor said this was sufficient to justify bringing in their best people from out of town and working extra shifts. In other projects in this study, the contractors said that the incentives were not large enough to make a difference.
4) The contractor was very experienced. This supports data from the benchmarking study (Chapter 2) and from other bridges in this study. Contractor experience makes a large difference.

5) There was good partnering and change management. From the post-construction meeting, it was clear that there was a spirit of co-operation between ODOT and the contractor, even though there was only one formal partnering meeting. A comment was made that key personnel from ODOT and the contractor often worked 100 hours a week and “slept in the trailer” so that they were available to make key decisions. The contractor also noted that ODOT was willing to accept reasonable changes in the specifications and that changes and approvals were turned around very quickly.

One interesting aspect of the post construction meeting was a discussion of how much ODOT should specify. The contractor expressed the view that ODOT should “tell us to build a bridge from A to B and let us figure out the best method.” The contractor stated that prescribing certain methods or designs might level the bidding playing field, but would probably not result in the lowest price or most innovative option.

The only real problem reported was with the grout. Grout was a problem on all the projects studied. The grout was used to level the concrete surface under the prefabricated steel piers. The contractor noted that the grout was not easy to work with. During the project, the grout shrunk away from the bottom of the steel pier and the gap had to be epoxy injected.

4.5 LOAD TEST

A load test was performed on PIC-22 in April 2004. The purpose of the test was to determine the load distribution factors and to determine if the design had been successful in achieving continuity. For brevity, only the outline of the test and the important results are presented here. A complete description of the tests and all results can be found in Lin (2004).

PIC-22 was loaded with two, standard ODOT tandem axle dump trucks (Figure 4.7). One truck weighed 59k and the other 51.7 kips. The distance from the front axle to the first tandem axle was 14 ft – 8 in and there was 4 ft – 10 in between the tendem axles. Four tests were conducted:

1) A moving load test in the westbound lane, single truck moving west.
2) A moving load test in the eastbound lane, single truck moving west.
3) A static test with two trucks, one in each lane, midspan of span #5 (second span from east abutment) (Figure 4.8).
4) A static test with two trucks back-to-back in one lane, midspan of span #5 (second span from east abutment) (Figure 4.9).

The steel girders in spans 5 and 6 (the two eastern most spans) were instrumented with strain gages. Figure 4.10 shows the placement of the gages on the bridge.
Figures 4.11 and 4.12 show the comparison between the theoretical AASHTO LRFD distribution factors and the measured distribution factors. Recall that the AASHTO LRFD distribution factor is the conservative, maximum value of load the beam will see. These factors will not add up to 1. The proper comparison is that none of the measured distribution factors exceed the calculated value, not if the measured distribution factors match beam-for-beam. The data show that, for one lane loaded, the experimental results are below the AASHTO values, but not by much. The maximum measured value is approximately 37% while the calculated value is 46%. The negative value for Girder 5 indicates uplift. Girder #5 is under a sidewalk.

For two lanes loaded, the difference is much greater. The AASHTO factor is 67%, while the test shows values of approximately 25-26%. The distribution in Figure 4.9 suggests that the girders under the roadway are sharing the loads almost equally. However, it should be noted that the trucks used in this test were small. Larger trucks may produce different results.

To check continuity, gages were placed over pier #5 (pier furthest east). The truck was then driven in one of the lanes, always westbound. The truck position was monitored and the strain in the girders was plotted as a function of truck position. This creates an influence line. The measured influence line can then be compared with the theoretical influence line.

Figures 4.13, 4.14, 4.15 and 4.16 show examples of the influence lines. The experimental data is not always continuous because the trucks were small and the bridge is large, so the strain data are sometimes very small and unreliable. These data points have been eliminated. In general, the data show very good agreement with the assumption of continuity.
Figure 4.8 – Static Test Position, Two Lanes Loaded (From Lin – 2004)

Figure 4.9 – Static Test Position, One Lane Loaded (From Lin – 2004)
Figure 4.10 – Distribution factors – two lanes loaded (From Lin 2004)

Figure 4.11 – Distribution factors – one lane loaded (From Lin 2004)
Figure 4.13 – Influence line at girder line 2 – East of pier 5 (from Lin – 2004).

Figure 4.14 - Influence line at girder line 2 – West of pier 5 (from Lin – 2004).
Figure 4.15 – Influence line at girder line 3 – East of pier 5 (from Lin – 2004).

Figure 4.16 – Influence line at girder line 3 – West of pier 5 (from Lin – 2004).
4.6 INSPECTION OF THE BRIDGE AFTER 4 YEARS

This structure was inspected in October 2008, and was found to be in remarkably good condition. No deterioration of the deck or superstructure was noted. Some minor cracking in the parapets, possibly due to shrinkage and spalling of coatings on the parapets near the deck were observed. The research team made a concerted effort to investigate the diaphragm area where the simply supported girders were embedded in concrete to make them continuous for live load to see if there was any cracking in this area. None was observed. No deterioration was noted in the piers.
CHAPTER 5
BRIDGES CLI-730-1113 AND MOT-70-1474
POST-TENSIONED ADJACENT BOX BRIDGES

5.1 INTRODUCTION

One objective of this project was to address the problem of bridge construction being the bottleneck in highway construction projects. One solution to this problem is the use of precast/prestressed elements with integral wearing surfaces. Ohio has used adjacent box girder structures for many years. Usually, the structure is a non-composite box structure, where the adjacent boxes are covered with a waterproofing membrane and then a layer of asphalt. Construction of the asphalt wearing surface takes additional time. The asphalt also tends to trap salt laden water against the top of the concrete boxes, leading to rapid deterioration in places where the membrane has failed or is damaged. The need for the asphalt surface could be eliminated by increasing the thickness of the top flange of the box girder and using this increased thickness as the wearing course.

Another problem with adjacent box girder bridges is leakage of the joints between the members. These joints, called shear keys, are meant to transfer shear forces between adjacent members. The origins of the shear key are not clear. The current design comes from early, two-stemmed precast members. The members were a channel shape with the opening facing down. They were connected by forming indentations in the side of each member. When placed side-by-side, the indentation formed a pocket which could be filled with grout and would lock the girders together. The key was at the top of the girder so it was adjacent to the flange. Later, the channel shape was made into a box by adding a bottom flange. This improved both the flexural strength and the torsional rigidity of the member. The shear key was never moved.

The shear keys tend to crack and leak. Research (Hucklebridge et al. 1995; Hlavacs, et al. 1997; Miller et al., 1998; Dong 2002) has shown that cracking is caused by perpendicular tensile forces which occur due to temperature and truck loading.

Further research (Hlavacs, et al. 1997; Miller et al., 1998; Dong 2002) has shown that the performance of the key can be improved by: Moving the indentation to the mid-height of the girder, using full depth shear keys, using improved materials and laterally post-tensioning the girders.

The use of lateral post-tensioning improves shear key performance by pre-compressing the grout in the key joint. The AASHTO LRFD Specifications require the use of lateral post-tension in order to use the most favorable distribution factor. The LRFD Specifications require that the post-tensioning provide a stress of 250 psi, but it is not clear if the means 250 psi along the entire side of the girder or just in the joint.
Several states use lateral post-tensioning for adjacent box girder bridges. However, Ohio has not used this design. As part of Strategic Initiative 9, Ohio DOT decide to build two, laterally post-tensioned box girder bridges. One would use an integral wearing surface and the other would use a cast-in-place composite wearing surface.

5.2 THE BRIDGES

5.2.1 Clinton-730-1113

CLI 730 is located on state route 730 over the Lytle Creek in the Town of Wilmington, just west of I71, halfway between Columbus and Cincinnati. The original structure, built in 1953, was a continuous concrete slab with capped pile abutments and piers and a ¾” monolithic concrete wearing surface. The replacement structure was single span, precast, prestressed concrete non composite box beam on reinforced concrete integral abutments. The beams were 94 feet long with an integral wearing surface and were designed under the old AASHTO Standard Specifications for HS-20 or the alternative military loading.

The innovations implemented in this project are:

- Box beams with an integral deck precast into the beam, with variable thickness top flange/deck to account for camber and a sag vertical curve
- Box beams are transversely post tensioned to improve key way joint performance.
- Final profile grade is achieved with surface grinding of the precast integral deck.

Six companies bid for this job and the contract was awarded in May 2003 with completion scheduled for October 2003. The construction was to be completed in two phases with the bridge kept in operation as a signalized single lane. This project also included all grading and drainage to construct the replacement structure.

5.2.2 Montgomery- 70-1421

MOT 70 was built over I-70 just north of Dayton, OH. The original bridge, built in 1956, was a continuous steel beam with reinforced concrete deck and substructure. In 2002, the bridge was struck by a truck and so seriously damaged that removal was required. The new structure was a two-span, prestressed concrete box girder bridge with a cast-in-place composite deck. Semi integral abutments and reinforced ‘T’ type piers were used. Span lengths were 73 feet and 110 feet. The bridge was designed under the old AASHTO Standard Specification as continuous for live load with an HS25 loading. The innovative feature of this bridge was the use of lateral post-tensioning.

5.2.3 Comparison of the Bridges

Both bridges used transversely post-tensioned, prestressed box girders. However, this was the only common feature. The differences between the two bridges were:
1) CLI 730 was a single span; MOT-70 was a two-span, continuous for live load structure.
2) CLI-730 had an integral wearing surface, which would be ground to profile. MOT-70 used a cast-in-place, composite surface.
3) CLI-730 was designed to use post-tensioning bars for the transverse post-tensioning, MOT-70 used strands.
4) CLI-730 was constructed in two phases, MOT-70 was not.

One important similarity between the two projects was that the same contractor won the bids for both jobs.

5.3 CLI-730-1113
5.3.1 Preconstruction

The preconstruction meeting held almost a month before the start of construction and was attended by representatives from ODOT, and the contractor. The contractor set a tentative start date of June 12th 2003 with a final completion date of October 3rd 2003. An interim date of completion was set for September 3rd.

An informal partnering agreement existed between the contractor and the contracting agency. It was decided at the pre-construction meeting that the administration and dispute resolution process would be based on the partnering approach. The dispute resolution and the claims processes were discussed and the accepted procedure was agreed on. At the meeting it was decided that grinding the top of the beams to profile would be done once all the beams are put in place rather than doing it in phases. The post tensioning company was of the opinion that the way the beams would be grouted, it wouldn’t leave much space for the post tensioning in phase two. Thus the design was slightly altered to loosen the tensioning.

One problem which came to light in the preconstruction meeting was the size of the beams. The original intent of the design was simply use the normal, noncomposite girder section and add a ½” wearing surface. This would have made the top flange 6” thick. Instead, the design engineer elected to use the top flange thickness of the standard composite section. The composite section has a 3.5” top flange and a 4” cast-in-place slab, making the minimum top flange thickness 7.5”. Because the bridge was at the bottom of a vertical sag curve, the flanges at the ends of the girder were thicker. This increased thickness raised the girder weight enough to make the girders permit loads. It was noted that two cranes would be required for placement of beams. The decision was made to close to the road to traffic during the erection operation.
5.3.2 Fabrication and Construction

Due to the unusual nature of this bridge, there was a higher level of communication between ODOT, the contractor and fabricator than normal. The role of experience was apparent in the early stages of fabrication. ODOT had constructed another integral wearing surface, laterally post tensioned bridge in Guernsey County. On the Guernsey County bridge, no one had accounted for lifting devices and grout ducts left protruding from the wearing surface. On this previous job, the contractor had to chip around the protruding elements so that they could be cut of below the surface and grouted over. For the Clinton bridge, the fabricator made dish-outs around protruding elements.

The plans for both the Guernsey and Clinton bridges required that pre-construction mock-up assemblies be made in the precasting yard. This was to assure alignment of the post-tensioning ducts. However, with the Guernsey bridge the mock-up was done on regular dunnage which did not account for the slope and crown in the actual abutments. Although the beams for the Guernsey bridge appear to align correctly in the yard, there were alignment problems during field assembly. To counter this, the mock-up for the Clinton bridge was to be done on mock abutments, which had the correct slope and crown.

Because the construction was phased, the contractor was concerned about differential camber at the construction joint. To counter this, the fabricator and contractor attempted to control the camber of the phase II beams. After the mock-up was approved, the phase I beams were shipped to site, but the phase II beams were left in place on the mock-up abutments. The contractor installed the phase I beams, and then measured the camber at weekly intervals – always on the same day of the week at the same time. This camber was transmitted to the fabricator who then attempted to adjust the camber of the phase II beams with addition or removal of dead weight.

The use of the integral wearing surface created additional work for the fabricator. Due to the vertical sag curve, the top of the boxes had to be curved. The fabricator had to extend the forms by welding on a metal plate.

In spite of all the planning, a problem was found during the mock-up. The post-tensioning contractor had supplied thin walled PVC ducts for the bars. This is common. However, during fabrication, the ducts floated due to the hydraulic pressure of the wet concrete. As a result, although the ends of the ducts were in the proper places, there was a noticeable curve to the ducts in each girder. In the mock-up phase this did not appear to affect the placement of bars.

The first construction problem occurred during placement of the beams. It was necessary for the contractor to align the post-tension ducts during placement. This is difficult to do with heavy beams which required 2 cranes to place. The situation was further complicated because the bridge uses integral abutments. Construction of integral abutments requires that reinforcing bar protruding from the abutments and the ends of the
girders be meshed during placement. Interference between the bars made it difficult to make the small adjustments in the beam position needed to align the PT ducts.

The purpose of the lateral post-tensioning was to compress the keyway and prevent cracking. This required the keys to be grouted before the post-tensioning was done. Since the bars must pass through the shear keys, it was necessary for the ducts to be sealed at the shear key. ODOT recommended the use of large, donut gaskets used by Michigan DOT. The contractor elected to use a thinner gasketing material. ODOT personnel did not require the thicker gaskets as this was considered a “means and methods” issue rather than a material specification issue.

During placement it was found that the beams had slight lateral deflections (sweep). This sweep was not out of tolerance, but over a 70 foot long beam, the gap at places between the beams was as much as \( \frac{1}{2} \) wider than expected. The contractor has to add additional gasket material at these places. There was also a problem with getting the beams to “snug up”. The contractor suggested that a small post-tensioning force be put on the structure prior to grouting to pull the girders together, but ODOT did not allow this.

Additional problems occurred during the shear key grouting operation. The bridge used full depth shear keys which were created by using the normal shear key design and leaving a 1 inch gap at the bottom. The contractor filled this gap with foam backer rod glued to the side of the beams with construction adhesive. This proved to be inadequate and there were numerous blow-outs during the grouting operation. However, the bigger problem was the PT ducts had not been properly sealed and shear key grout leaked into the PT ducts. The contractor had been required to put the PT bars into the ducts before grouting the shear keys to verify alignment. Unfortunately, the bars were left in place and some of the bars were grouted into place prior to being post-tensioned. This was discovered when the contractor attempted to post-tension the bars and some of the bars did not have the required elongation at the required tension. An attempt was made to free the stuck bars by loosening the dead head anchorage and applying the PT force. This only worked in isolated cases. The final post-tensioning results are summarized in Table 1.

### Table 5.1 - Phase I Post tensioning summary

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<thead>
<tr>
<th>LOCATION</th>
<th>TOP BARS</th>
<th>MIDDLE BARS</th>
<th>BOTTOM BARS</th>
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Most of the difficulties on Phase II were a direct result of the problems encountered on Phase I. The plans called for the post-tensioning bars to be coupled at construction joint. This is done with a standard mechanical coupler. Because many of the phase I PT bars had inadvertently been grouted in place prior to being post-tensioned, their final position could not be readjusted to assure alignment with the phase II couplers. The alignment of the phase II bars was also uncertain.

The engineer had provided a 2 inch diameter duct for the 1 3/8 inch diameter bars. There was some minor misalignment of the ducts of adjacent members due to differential camber. The ducts had floated when the beams were cast, so the ducts were not perfectly straight. While these factors did not prevent the bars from being placed through the beams, it greatly limited the ability of the contractor to align the phase II bars with the phase I bars. ODOT requested that the ducts on the phase II beam be cored to a larger size to allow adjustment of the PT bar positions, however, when this was tried it was found to be impractical. ODOT decided to simply leave things as they were and see how many bars could be coupled. After assembly of phase II, it was found that 24/27 bars could be successfully coupled. The 3 bars which could not be coupled to phase I were anchored at the construction joint. The results of the post-tensioning are shown in Table 2.

**Table 5.2 - Phase II Post Tensioning Summary**

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In the end, the bridge did not perform as expected. Cracks occurred in the shear keys, especially in phase I. As a result, some of the joints leaked when it rained. This bridge was constructed by ODOT, but was to be maintained by Clinton County. ODOT did not want to give the county a bridge with a maintenance problem, so a waterproofing membrane and an asphalt wearing surface were added.

**5.3.3 Post Construction**

A post construction meeting was held after the completion of the construction of CLI-730. It was attended by representatives from ODOT, the contractor and the sub contractors. Among the important points raised were:
1) There was no pre-bid meeting. This led to a situation where the contractor felt they did not have sufficient information about certain parts of the project during bidding. Specifically, there were a lot of notes and addendums, which were often found to be confusing. The ability to discuss and get clarification would have helped. The contractor also stated they were unaware this was a pilot project until the preconstruction meeting.

2) The contractor noted that there were some procedures where guidance would have helped – specifically controlling camber, a method for casting the center joint and general method for casting the full depth shear keys.

3) There were delays getting drawings from subcontractors and delays getting approvals from ODOT. A streamlined procedure is needed. One of the problems noted was that there are a limited number of post-tensioning contractors. Because of this, the contractor had a limited ability to push the subcontractor as there were few, if any, alternatives.

4) There were problems with the grout – a recurring theme in all SI-9 projects.

5) There were difficulties with the constructability. As previously noted, the girders were too heavy. There was steel embedded in the end of the girders which connect to the abutments. However, this steel got in the way when the contractor tried to pull the beams together to seal the joints. There were problem with the holes for PT bars not being aligned and not being large enough to allow for adjustment. The difficulties appeared to be from a lack of experience with this type of bridge.

6) While an informal partnering agreement existed, the contractor stated that more meetings would have been helpful.

7) The contractor noted that it was difficult to find qualified PT grout technicians.

5.3.4 Final Inspection

This structure was surveyed in February 2009, and was found to be in remarkably good condition. The temperature on that day ranged between 20 and 30°F. No deterioration of the deck or superstructure was noted. The only thing worthy of noting was some cracking asphalt at the ends of the structure, possibly due to thermal expansion and contractions.
5.4 MOT-70-1471

5.4.1 Preconstruction and Fabrication

Construction of the Montgomery County bridge began while the Clinton County bridge was being constructed. The same contractor won the bid for MOT-70, but a different precaster was used. Using lessons learned from the CLI-730 bridge, the following changes were made for MOT-70

1. **Duct Alignment and Size:** PVC ducts were used in the beams for CLI-730. These ducts floated when the beams were cast, causing them to be slightly curved. This made it difficult to push the PT bar through all of the beams. For MOT-70 the precaster still used PVC ducts, but used temporary metal sleeves to keep the ducts straight. These sleeves were removed after casting. The ducts were also oversized to allow for adjustment of the PT strands in the final bridge.

2. **Mock-up and Camber Control:** In the GUE-513 bridge, the engineer asked for a mock-up assembly, but the mock-up was done without considering the crown of the bridge. As a result the beams appeared to align properly in the precast yard, but did not align properly in the field. The mock up tests for MOT-70 were conducted with the slopes of the abutments accurately replicated in the yard. The mock up was also used to monitor camber. If the camber became too large, the precaster placed dead weights on the beams to control it. The precaster for MOT-70 made two beams together and all the beams on the same bed in the yard. This resulted in more uniform beams and there were fewer problems with camber.

3. **Sealing P/T Ducts** After the problems with CLI-730, the contractor was very cautious about sealing the post-tensioning ducts. In CLI-730, ODOT personnel were reluctant to give guidance to the contractor. This however changed at the MOT-70 project. ODOT worked closely with the contractor and ODOT personal also assumed a more supervisory role to ensure that the desired “doughnut seals” were used, were sealed carefully and that no gaps were left when the beams were being positioned. As with CLI-730, the sides of the beam were not perfectly straight, making sealing the gaps between the adjacent beams difficult. The contractor attempted to use come alongs to bring the beams together. This did not work. Unlike CLI-730, ODOT allowed a slight pre tensioning of the lateral post tensioning tendons to seal the joints.

4. **P/T Strands** Strands were used for post tensioning instead of the bars that were used at CLI-730. It was noted that the strands were easier to push through the sleeves than the bars. The use of the metal sleeves during casting kept the ducts straight and this helped in providing a clear straight channel to push the strands through. Moreover, since the strands were lighter than the bars and could be bent slightly it was easier to work with them.
5.4.2 Construction

Unlike the Clinton bridge, construction of the Montgomery bridge was uneventful, except for some transportation issue. The precast pieces were assembled easily. The post-tension issues were resolved by changes to the design using the lessons learned on the Clinton bridge.

One of the transportation problems was that there are only a limited number of trucks available transport the girders from the yard to the site. This makes the schedule heavily dependent on the availability of these trucks. Ideally the contractor would have like to bring all the beams at one time and erect the entire structure at one time. However, this could not be done and the erection took two days.

Another problem occurred with permits. The permits did not allow transportation of the beam during the evening hours, so the beams had to be moved during the day. The idea was to move the beams during the day and erect the beams at night. On the first day, one of trucks broke down along the way. The local police disallowed a request to transfer the beam on to another truck and bring it to the site because the permit was tied to a specific truck. Once the situation was resolved, it was evening and the beam could not be transported until the next day. An entire evening’s work was lost. The rules did not allow work on weekends over a busy interstate like I-70, but special permission was sought and later granted to make up for the lost work.

5.4.3 Post Construction

As with CLI-730, there was a post-construction meeting for MOT-70. The important points raised were:

1) No incentive was offered for this project and the contractor observed that there was no encouragement to complete this job on time.
2) In the CLI-730 project, the turn around time for the shop drawing was long; usually more time than necessary. However, in the MOT-70 project ODOT personnel made sure the drawings were reviewed and returned early to keep the work was on schedule.
3) It was suggested that ODOT consider purchasing the beams early or maybe give out another contract for the beams so that the contractor does not have to wait for the fabrication of the beams.
4) The use of stands rather than bars was considered beneficial. It was easier to put the strands though the sleeves even though more than one strand was pushed through. The strands were more flexible and it could be twisted through the sleeves unlike the bars.
5) Making all of the beams on the same bed and two at a time kept the camber under control.
6) Allowing the beams to be brought together with a slight bit of pre tensioning on the lateral tendons was considered necessary to seal the ducts.
7) As with all SI-9 projects, problem with mixing and placing the grout were noted.
8) The contractor raised a concern about the shear key design. Full depth shear keys were specified, but no construction method was given. The contractor used backer rod, which proved inadequate. This was a concern because the bridge is over an Innerstate and grout fell on cars passing underneath.
9) Permits issues and the lack proper transport were discussed. The contractor noted that this issue could have been resolved if on-site storage had been available.

5.4.4 Final Inspection

This structure was surveyed in February 2009, and was found to be in remarkably good condition. The temperature on that day ranged between 20 and 30°F. No deterioration of the deck or superstructure was noted. The only things worthy of noting were some cracking in the parapets, possibly due to shrinkage, spalling of coatings on the parapets, and the expansion joints in the parapets at the ends of the structure were open, apparently due to the low temperature and thermal response of the structure.

5.5 OBSERVATIONS OF THE RESEARCH TEAM

CLI – 730 was the only unsuccessful bridge done in this project. However, it proved to one of the most instructive. The problems with CLI-730 were not technical. Instead the problems were related to a lack of experience and a lack of true partnering.

At several points, lack of experience played a key role in the problems experienced. As previously noted, the engineer used the composite beam + 4 inch deck rather than the non-composite beam + ½ wearing surface. When the build up at the ends of the beams (for the vertical sag curve) was added, the beams were so heavy that 2 cranes were needed for erection. The precast contractor had problems with the post-tensioning ducts displacing during pouring, resulting in a misalignment. Although ODOT personnel had researched methods for sealing the joints between the beams (and chose to use donut shaped gaskets), the contractor elected to use a different type of gasket. Since this would be ‘specifying means and methods’, no one objected. The contractor’s choice of the type adhesive to hold the gaskets in place also proved ill-advised as the seals fell off during erection. Neither ODOT personnel nor the contractor realized that normal variations in the sides of the beams would prevent proper sealing and the design of the bridge made it difficult for the contractor to pull the beams together. ODOT would not allow the contractor to apply some post-tensioning to pull the beams together. The result was a failure of the gaskets. Once the gaskets failed in Phase I, grout leaked into the post-tensioning ducts making post-tensioning impossible.

These problems were corrected in Phase II, but that left half the bridge without proper post-tensioning. A coupling system had devised join the PT bars from Phases I and II, but no one realized that any misalignment of the bars in Phase I made joining the bars from the two phases impossible.
The problems listed were not due to incompetence or a lack of due diligence. Instead, they were largely due to a lack of experience. The success of MOT -70 using the lessons learned on CLI-730 is telling.

However, the bigger problem with CLI-730 was that the contractor might have benefitted from more direction from ODOT. When questioned about this, ODOT field personnel noted that they were prohibited from specifying “means and methods”; in other words, they could not tell the contractor what to do. The reason given was that if ODOT specified the means and methods, then ODOT would incur a liability if anything went wrong. However, one senior ODOT engineer observed that the only real liability would be the inability to withhold some damages from the contractor. This same engineer further observed that a few dollars in damages is poor compensation for a bridge with an unsuccessful outcome.
6.1 INTRODUCTION

The AASHTO TIG on rapid construction highlighted the use of prefabricated deck panels as a means of decreasing construction time. On bridge HAN-75-15.99, ODOT used precast, post-tensioned concrete bridge deck panels. It was thought that using precast panels would save construction time by eliminating the need to form a deck. Deck forming can be eliminated through the use of stay-in-place forms (SIP), but ODOT did not want to use SIPs in this application. HAN-75-15.99 is over I-75 and there were concerns that the forms could deteriorate and a piece could fall to the roadway below. There were also concerns over the fact that the bottom of the concrete deck could not be seen or inspected through the SIPs.

The use of precast/post-tensioned deck elements was a more acceptable solution for this application. In addition to shortening construction time, it was hoped that the longitudinal post-tensioning would eliminate cracks in deck as deck cracking is a major problem with cast-in-place decks.

6.2 DESCRIPTION OF THE BRIDGE AND PANELS

Bridge HAN-75-15.99 over I-75 in Findlay, Ohio. This bridge is a 170 foot single span steel plate girder bridge. The deck is comprised of 16 precast, prestressed concrete panels, each 10 foot, 3 inches long, 41 feet wide, and 10.75 inches thick, placed transversely on the girders (Figures 6.1 and 6.2).

The panels were fabricated at the plant as reinforced elements and post-tensioned in the panel’s longitudinal direction before being shipped to the site. The post-tensioning ducts which are longitudinal to the panel end up being transverse to bridge girders when the panels were placed on the bridge. Shear key joints were cast in the sides of the panels and these joints were grouted before the final post-tensioning was done. The panels were then post-tensioned together (longitudinally to the direction of the bridge; transverse to the panels). After post-tensioning, shear studs were welded to the top of the steel girders through pockets left in open in the panels. The panels were attached to the girders by grouting the shear studs into the pockets.
6.3 DETAILS OF THE PANELS

Details of the concrete deck panels are shown in Figures 6.3 and 6.4. The deck panels are 10.75 inches thick have four post-tensioning ducts that run longitudinal to the panel (transverse to the bridge). These ducts contain four 0.6 inch diameter uncoated seven wire strands which were tensioned and grouted before the panels left the fabricator’s yard (Figure 6.5). The concrete was to be at least 5500 psi at this time. The maximum jacking force per tendon was specified 45k/strand or 180 kips total for the 4 strand tendon. For two tendons in one panel, a friction test was performed by placing a load cell at the deadhead (unjacked end). The friction loss was found to be approximately 6-9%. Seating loss, as measured by the change in elongation during the seating process, was 2-4%.
There are 12 transverse (longitudinal to the bridge) post tensioning ducts in each panel. After assembly of the panels on the bridge, four 0.6 inch diameter uncoated seven wire strands were pushed into each these ducts and tensioned. The tensioning procedure is explained in a later section. To tie the panels to the steel plate girders there are 20 (4 rows of 5) shear stud pockets with 9 studs per pocket per panel (Figures 6.3, 6.4 and 6.6).

Figure 6.3 Typical precast concrete panel (Mild steel omitted for clarity)

Figure 6.4 Typical panel before casting
It was specified that each panel be at least 45 days old at erection to allow the panels creep and shrink. This helps to minimize loss of longitudinal (to the bridge) prestressing forces. The top surface of the panels would later be ground to profile, there is no overlay on this bridge. Prior to shipping the panels to the project site, the entire deck was assembled in the yard. This was to verify that all the deck panel units were constructed in compliance with all the plan requirements. Blocking and bedding strips were used to simulate the support of the deck panels on the beam top flanges. Although the deck panels were checked for proper fit-up and alignment, during placement of the
panels on the girders the contractor had to go back twice and switch the panels because of differential camber.

6.4 CONSTRUCTION SEQUENCE

The previous bridge was a 3 span continuous steel beam bridge carrying 2 lanes of traffic. Because the detour was short, the bridge was closed for construction. The expected closure was 120 days starting in July 2004. Once the bridge was closed to traffic the superstructure and abutments were demolished and removed. The existing piers remained until the new bridge was erected.

After the MSE abutment walls and wing walls were constructed, the 78 inch deep steel plate girders were shipped and set into place. The girders were placed at night while I-75 was closed intermittently using the abandoned piers as a launch point. This eliminated one splice which in turn decreased erection time. Once the girders and cross frames were in place, the precast concrete deck panels were placed on the girders (Figure 6.7).

As with the girders, I-75 was closed intermittently to place the panels. This operation took two nights to complete. Styrofoam shims were placed on top of the girders (these would later serve a forms for the grout haunch) and leveling bolts were used to achieve the required profile grade. Figure 6.8 shows the bridge after the completion of the panel placement.
Before the grout was poured in the joints the post-tensioning ducts between panels were connected. This was also the time that the vibrating wire strain gauges were placed in the joints, (see Experimental Program). It took two days to grout all of the joints due to rainy weather conditions. Once the joints were grouted, the post tensioning strands were installed into the longitudinal ducts (Figure 6.9).

The bridge panels were post-tensioned longitudinal to the bridge (transverse to the panels) when the shear keyway grout, anchorage blisters, and deck panels had all achieved a minimum of 6,000 psi compressive strength. A post-tensioning sequence was provided on the drawing, but as will be discussed later, the contractor did not follow it. Tendons were tensioned from one end of the bridge, with a dead head used at the other end. To minimize the effect of friction loss, the contractor was required to switch ends after tensioning each tendon; that is, odd numbered tendons were all tensioned from the
same end of the bridge and even numbered tendons were tensioned from the opposite end.

Three days after post tensioning the longitudinal ducts were grouted. Nine Nelson studs were installed in each deck panel pocket. Four days after this the diaphragms, shear stud pockets, and the void between the top of the girder and the bottom the deck panels were grouted. At the point the bridge was completely tied together.

The approach slabs were then constructed and the deck was ground to profile, a minimum of ¼ inch in the longitudinal direction between the faces of the sidewalk curbs. The rest of the items: pouring sidewalk and parapet, earthwork, fence installation, and sealing the entire concrete deck and approach slabs between faces of sidewalk curbs were then completed.

6.5 EXPERIMENTAL PROGRAM

One of the common problems in adjacent structures is the leaking of the shear keys. This causes premature deterioration of the structure. Post-tensioning compresses the keys so that cracking and leaking is avoided or minimized. To this end, the experimental program had 3 goals:

1) To visually monitor the joints for signs of cracking/leakage.
2) To use instruments in the joints to try to determine:
   a. Initial post-tensioning stresses in the joints;
   b. Loss of post-tensioning force over time;
   c. Thermal stresses.
3) To periodically load test the bridge to assess the composite action of the precast panels.

It would have been desirable to instrument the panels, but the panels had already been cast by the time the decision to instrument the bridge was made. As a result, only the steel girders and the panel joints could be instrumented.

Vibrating wire strain gauges were placed in the joints between the precast deck panels (Figure 6.10 and 6.11) prior to grouting the joints. The gages were held in place by a tapered plastic support and wire (Figure 6.10). Once the grout was placed and set the plastic holder pieces were removed and any exposed was wire cut. The tapered holes were then plugged with grout to prevent long term deterioration of the joint.
Figure 6.10 Vibrating Wire Gage placed in the joint

Figure 6.11 Keyway joint details

Figure 6.12 Vibrating Wire Gauge Layout
A total of 15 gages were placed in the joints, as shown in Figure 6.12. In addition, vibrating wire gages were glued to the top and bottom flanges of the two interior girders (Figure 6.12). These gages were placed at the quarter point as it was the furthest place which could be reached safely and it was not over the roadway where a falling gage could damage a car.

All of the cables were threaded through PVC pipe to the sidewalk. Another pipe was poured into the sidewalk and turned out at the southeast side of the abutment. The CR10X data acquisition system was then attached to an existing pole at the end of the southeast wing wall.

### 6.5.1 Measurements during grouting

The grouting of the transverse joints took place from west end of the bridge to the east end of the bridge over a two day period. Grouting was hampered by rainy weather conditions. In the first day two of the joints that contained vibrating wire gauges were grouted and set overnight while the rest were poured on the next day. Figure 6.13 shows the average response of the gages in each. All gage readings have been corrected to remove temperature effects on the gages themselves. Initially, the gages show expansion due to heat of hydration and expansion of the grout. Then, the gages show some reduction in strain due to cooling and grout shrinkage (non-shrink grout does shrink, it simply expands first such that the net movement is nearly zero).

![Figure 6.13 Average strain in each joint during and after grouting the joints](image)

Figure 6.13 Average strain in each joint during and after grouting the joints
6.5.2 Measurements during post-tensioning

The post-tensioning started nine days after the joints between the panels were grouted. During those nine days the forward ends of the girders were jacked up to readjust the bearing devices. As the panels were not yet tied to the girders, this did not appear to cause any changes in the panel behavior. During this time, the grouting valves and post-tensioning strands were installed in the longitudinal ducts. It took two days to stress all of the tendons. Figure 6.14 shows the longitudinal tendon stressing sequence from the drawings and the actual stressing sequence. It is of note that the research team was monitoring the gages and not paying particular attention to the stressing sequence. The research team assumed the sequence on the drawings was used, but this could not be correlated to the data. From the data, the research team determined 3 possible stressing sequences and ODOT later confirmed that one of these three possibilities had, indeed, been the actual sequence. This confirmed the robustness of the data. No reason was given as to why the sequence was altered. However, the difference between the two sequences is not significant enough to think that panel performance might be compromised.

![Longitudinal tendon stressing sequence](image)

Figure 6.14 Longitudinal tendon stressing sequence

Table 6.1 shows the change in strain which occurred when the center, far north and far south tendons were tensioned. Table 6.2 shows the final TOTAL strains developed in the joints.
Table 6.1 Change in Strain (in microstrain) in Each Gauge Due to Post-Tensioning

<table>
<thead>
<tr>
<th>Gauge Location</th>
<th>Location of Tendon (Tendon Stressed)</th>
<th>North (4)</th>
<th>Center (1)</th>
<th>South (6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 – North</td>
<td>27.9</td>
<td>3.6</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>A2 – Center</td>
<td>5.3</td>
<td>33.1</td>
<td>7.9</td>
<td></td>
</tr>
<tr>
<td>A3 – South</td>
<td>2.7</td>
<td>8.5</td>
<td>26.5</td>
<td></td>
</tr>
<tr>
<td>B1 – North</td>
<td>21.8</td>
<td>7.1</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>B2 – Center</td>
<td>14</td>
<td>11.6</td>
<td>8.4</td>
<td></td>
</tr>
<tr>
<td>B3 – South</td>
<td>4.7</td>
<td>12.8</td>
<td>17.3</td>
<td></td>
</tr>
<tr>
<td>C1 – North</td>
<td>18.3</td>
<td>7.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>C2 – Center</td>
<td>13.2</td>
<td>8.5</td>
<td>12.7</td>
<td></td>
</tr>
<tr>
<td>C3 – South</td>
<td>1</td>
<td>11</td>
<td>19.4</td>
<td></td>
</tr>
</tbody>
</table>

**Note that joint D is similar to B and joint E is similar to A**

Table 6.2. Total Strain (in microstrain) Developed by Post-Tensioning

<table>
<thead>
<tr>
<th>Gauge</th>
<th>A1</th>
<th>A2</th>
<th>A3</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>D1</th>
<th>D2</th>
<th>D3</th>
<th>E1</th>
<th>E2</th>
<th>E3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before PT</td>
<td>2560</td>
<td>2634</td>
<td>2691</td>
<td>2605</td>
<td>2619</td>
<td>2660</td>
<td>2648</td>
<td>2503</td>
<td>2662</td>
<td>2503</td>
<td>2684</td>
<td>2611</td>
<td>2531</td>
<td>2640</td>
<td>2532</td>
</tr>
<tr>
<td>After PT</td>
<td>2415</td>
<td>2461</td>
<td>2544</td>
<td>2481</td>
<td>2482</td>
<td>2549</td>
<td>2348</td>
<td>2332</td>
<td>2561</td>
<td>2378</td>
<td>2471</td>
<td>2474</td>
<td>2395</td>
<td>2485</td>
<td>2434</td>
</tr>
<tr>
<td>Diff.</td>
<td>145</td>
<td>170</td>
<td>147</td>
<td>124</td>
<td>136</td>
<td>141</td>
<td>110</td>
<td>172</td>
<td>141</td>
<td>125</td>
<td>113</td>
<td>137</td>
<td>136</td>
<td>154</td>
<td>98</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Avg.</th>
<th>154</th>
<th>134</th>
<th>141</th>
<th>125</th>
<th>129</th>
</tr>
</thead>
</table>

There is no information regarding the elastic modulus of the grout used. Bags of the same brand of grout were obtained by the research team, mixed according to manufacturer instructions and tested. The grout was found to have a modulus of elasticity of 4,000,000 psi.

\[
P = 12 \text{ ducts} \times 180 \text{ kips} = 2160 \text{ kips total load}
\]

\[
f_{pt} = P/A = 2160 \text{ kips} / (41 \text{ feet} \times 10.75'' \times 12 \text{ in/ft}) = 0.408 \text{ ksi} = 408 \text{ psi}
\]

\[
\varepsilon = 408 \text{ psi} / 4,000,000 \text{ psi} = 100 \mu \text{ strain}
\]

This is not inconsistent with the strain values in the table. Also, recall that modulus of elasticity can easily vary \( \pm 20\% \) for cementitious materials. Finally, there were probably differences between the grout used on the bridge and the lab specimens (different lot, differences in mixing, different water contents, etc.). Note that if the grout is assumed to have a modulus of elasticity of 3,000,000 psi (not unreasonable for grout), the expected strain would be 134 microstrain; exactly the average value.
Table 6.2 shows differences in the strains both within a joint and between joints. The differences in strain could be attributed to many factors. The gauges could have shifted after placement before grouting so they might not have remained longitudinal. The material properties of each joint might vary. This variation is not uncommon with grout, which is mixed in small batches and, often, with variations in the amount of water added. There may also be a variation in stress in the joints caused by friction between the panels and the girders, friction losses in the tendons and variations in stress from the tensioning sequence. However, in spite of these minor differences, the joint stresses are consistent with expectations.

After post-tensioning, the panels were monitored for environmental effects. It was noted that the relationship between temperature and strain is reversed. When the temperature increases the strain decreases, signifying compression. The panels want to expand due to the increase in temperature but are restrained so they cannot move. This causes compression in the joints. Conversely, temperature decreases cause an increase in strain as the panels try to contract but are restrained.

The last step to tie the superstructure together is to grout the shear stud pockets and void between the top of the girder and the bottom of the deck panel. This occurred 18 days after the joints were grouted and 9 days after the panels were longitudinally post-tensioned. The strains in the joints didn’t significantly change after shear stud pockets were grouted.

6.6 CONTINUED MONITORING

The panel joints were monitored for environmental effects. After 17 months, there were no cracks in the joints. This is significant as ODOT and other states have seen cracking in cast-in-place desks within 8 months. Figure 6.15 shows typical results for a single joint over the monitoring period. All the joints showed almost identical behavior, so this graph is representative of all. There are several breaks in the data. At approximately one month after the post-tensioning the wires to this box were severed by the contractor digging a trench. This damage was repaired a month later when the research team was able to move the data system a new post in front of the southeast wing wall. In this process a gauge was damaged and no longer works. The remaining gaps occurred to problems with the data system which caused a loss of data. However, even with the gaps the trends are still clear.

The strain curve shows the typical “power curve” shape expected for creep and shrinkage induced changes. There are some “bumps” in the curve, the effect of daily and seasonal temperature changes. The change in strain is approximately 550 microstrain. Assuming the steel changes the same amount, the long term loss would be 15.7 ksi, or about 7%. The problem with this calculation is that the measurement was made in the joints, not in the panels. However, it does provide at least an estimate of the loss of prestressing force in the system.
 Approximately 5 months after the structure was post-tensioned a load test was performed. Four loaded trucks were placed on the bridge over the joints to determine the response and check for composite action. The trucks were not heavy enough to obtain a large response. The case of the four trucks placed over the center joint, two side by side then the pair back to back, yielded the greatest response (Figure 6.16).

Again using 3,000,000 psi for the modulus of elasticity the calculated strain at the center of the deck is approximately 16 microstrain. While the maximum strain (at the center gauge) is 36 microstrain (Figure 6.17), the average for the joint is 23 microstrain, which is consistent with the calculated value considering the small values of strain obtained. When the loads are placed at the quarter points, the calculated strain at the center joint is 8 microstrain. The measured average is 12 microstrain. Again, this is reasonable considering the small values of strain. The data suggests that the panels are behaving as part of composite system, as designed.
Figure 6.16 Truck configuration during load test

Figure 6.17 Strain in gages at the midspan (Joint C) during load test
6.8 POST CONSTRUCTION MEETING

A post construction meeting was held on February 14, 2005 to discuss the positive and negative aspects of this bridge. Overall the project was a success as the bridge was opened to traffic within 120 days of closure. Everyone involved in the project agreed that the communication system in place was the key to this type of construction. Open lines of communication were crucial for transporting the girders and panels to the job site. All of the affiliated agencies, including the Highway Patrol, were contacted in advance about transporting after dark. This was significant as problems had been encountered with transporting large precast members after dark in previous projects.

The project also showed the importance of clear and consistent specifications. The contract documents specified 2 companies whose products could be used for the post-tensioning ducts. The engineer, wishing to avoid leaks during the grouting operation, specified that a leak test was to be performed by pressurizing the PT ducts to 100 psi using air. However, the specified products did not have fitting capable of sustaining this pressure. This became a major issue which needed to be resolved between the engineer, the contractor and ODOT.

During the post-construction meeting, some items were identified which could be improved if this design was used in the future. The design and specifications on this project lead itself to a linear construction sequence after grouting the joints. For example, the grinding of the panels had to be done after the approach slabs were placed but before the sidewalks were placed. It was suggested that an overlay on the deck, instead of grinding it, would allow for a less linear schedule. Although actually grinding the panels was very straightforward and didn’t pose a problem, overlaying the deck would allow more than one task to be completed at once. Cast the sidewalk and parapet on the panels at the plant may also speed construction, but such panels would be more difficult to ship.

The precast concrete panels were much easier to install than forming and pouring a deck the conventional way. The cambers of the panels were within the specifications but while erecting the panels the contractor had to go back twice to shift them around. This was expensive but not difficult.

Future panels could be pretensioned as opposed to post-tensioned. For this particular project, the panels were post-tensioned to avoid special bed preparation. It was recommended that the panels be match cast so that no keyway is needed.

6.9 OBSERVATIONS OF THE RESEARCH TEAM

Overall this project and method of construction was successful. Based on the data obtained, the following conclusions can be drawn:

1) The bridge was completed on time under a very aggressive schedule. Although the there was not a time savings, with some changes to the design to
eliminate the construction schedule linearity precast concrete panels for bridge decks is a viable alternative to conventional construction.

2) The post-tensioning of the deck panels was successful. Data shows that the required stress was achieved in all of the joints. Eight months after grouting the joints between the panels the deck shows no signs of cracking. Cast-in-place decks often display transverse, full depth cracking by this time.

3) Data shows a contraction in the joints over the first 2 ½ months, probably due to creep and shrinkage of the panels and the joints. After this time, strain changes appear to be temperature induced.

4) Load testing shows that the panels are behaving as designed, as part of a composite system.

6.10 FINAL INSPECTION

The structure was inspected in March 2007. The deck was in generally good shape although one of the joints in the panels showed signs of deterioration, as did the expansion joint next to it (Figure 6.18). The joints appeared intact from the top (Figure 6.19). Many showed stains from leakage on the bottom. However, review of construction photos showed this staining was present before the joints were grouted and probably do not indicate a leak.

Figure 6.18 – Deteriorated deck joint and expansion joint.
Figure 6.19 – Intact joint from top

Figure 6.20 – Stains on panel bottom. These stains were present at construction before grouting and probably do not indicate a leak.
CHAPTER 7
BRIDGE FAI-22-15.85
MONITORING A POST-TENSIONED ADJACENT BOX GIRDER BRIDGE

7.1 INTRODUCTION

FAI-22-15.85 was the last bridge completed for this research. All of the bridges in this study, except PIC-22, were some type of adjacent, post-tensioned concrete structure. In the first 3 case studies, GUE-513, CLI-730 and MOT-70, the research team looked at the construction issues. For HAN-75, the team did some structural monitoring. For FAI-22, the research team looked at both construction issues and did some structural monitoring.

7.2 BRIDGE FAI-22-15.85

FAI-22 is an adjacent box girder bridge located on State Route 22 in Lancaster, Ohio. The bridge is located just to the west of the main business district in Lancaster and Rte 22 is a major east-west thoroughfare through the city. Phased construction was used for maintenance of traffic.

The bridge is 63 feet long, 56.3 feet wide and has a 15° left forward skew. It consists of sixteen modified ODOT B27-48 beams and one modified ODOT B27-36. The modification was that the top flange was increased from the normal 5 inches to provide an integral wearing surface. To accommodate the camber of the girders, the thickness of the integral wearing surface was increased at the ends. Thus, the top flange of the beams was 6 inches (5 inch top flange + 1 inch integral wearing surface) at midspan and it was gradually increased to 6 ½ inches at the ends. This wearing surface was ground to profile before the bridge opened.

The beams were laterally post-tensioned at six locations (Figure 7.1). To accommodate phasing, Phase I was post-tensioned at erection and opened to traffic. When Phase II was erected, the post-tensioning bars from Phase II were coupled to those of Phase I with a mechanical coupler and then the Phase II bars were tensioned. To assure proper bearing between the beams, full depth shear keys, grouted with standard non-shrink grout were used. Details of the bridge are shown in Figures 7.1 – 7.4.
Figure 7.1 - Plan View (Drawing courtesy of PSI LLC)

Figure 7.2 - Section (Drawing courtesy of PSI LLC)
This Spec of the unrea...e 2009)

over 7.3 Mod...There...si...issue (C4.6.2.2.1). However, given the size of the shear key and the geometry of the bridge, ODOT determined that adding additional prestressing bars would be unrealistic.

The problem is that there has never been an adequate justification for the 250 psi requirement in the AASHTO LRFD Commentary. It is widely believed that this value comes from segmental construction, so it is questionable as to whether this is applicable to box girder bridges. A recent paper in the PCI Journal (Hanna, Morcous and Tadros, 2009) provided updated guidelines for transverse post-tensioning. FAI-22-15.85 is just over 68 feet wide and the girders are 30 inches high. Interpolating Figure 6 in Hanna,
Morcous and Tadros paper, the required post-tensioning would be approximately 14 kips/ft. Since the bridge is 65 feet long:

\[
14 \text{kips/ft} \times (65 \text{ ft}) = 910 \text{kips} < \ (12 \text{ bars}) \times (131.2 \text{kips/bar}) = 1574 \text{kips}
\]

The lateral post-tensioning provided greatly exceeds the new recommendations, and therefore should be adequate.

**7.3 INSTRUMENTATION AND MONITORING DURING POST-TENSIONING**

**7.3.1 Instrumentation**

One objective of the study was to try to determine if the lateral post-tensioning is effective. To this end, the bridge was to be instrumented during the post-tensioning process. The intent of the post-tensioning is to compress the shear key joint to prevent cracking, so the instrumentation focused on the keys.

Since the object was to measure the stress in the shear keys, the research team employed pressure measurement cells. The cells (GEOKON Model 4810) are vibrating wire instruments usually intended to measure the pressure between soils and buildings. It was not certain whether these gages would prove effective, but it was the only instrument available which could be even reasonably considered.

Cut-outs were placed in the side of some girders during fabrication (Figures 7.5 and 6). The pressure cells were attached to the sides of the girders with screws. A spacer on the screws maintained a gap between the cell and the concrete surface. The area behind the cells was grouted with same non-shrink grout was would be used in the keyways.

![Figure 7. 5– Cutout in side of beam.](image)

![Figure 7. 6– Grouted cell](image)
After installation of the beams, the shear keys were grouted. During the post-tensioning process, external vibrating wire strain gages (VWG) were attached to the top of the bridge over the joints (Figure 7.7). These gages were installed the day before post-tensioning and left overnight to monitor thermal movements. The positions of all instruments are shown in Figure 7.8. Note that the bridge was constructed in phases. The VWGs were placed on the joints of Phase I during the post-tensioning operation and removed that same day. This was to allow for grinding of the deck and moving traffic to Phase I. Likewise, the VWGs on Phase II were removed a few days after post-tensioning to allow for completion of the bridge. The pressure cells are permanently embedded.
7.3.2 Results of Post Tensioning

The bridge was monitored during both phases of post-tensioning. During the post-tensioning, the pressure cells registered only 10 psi of pressure. It is not certain if this is a correct reading or not. However, it does not seem reasonable. It is believed that the problem was that the gages are really meant for soil and the grout was simply too rigid to properly compress the flexible top face of the gage.

The vibrating wire gages did work. Figure 7.9 shows typical responses of a vibrating wire gages (marked with circles in Figure 7.8). In Phase I, the gages shown are over Bar #3 and in Phase II, they are over Bar #2. The results are only shown during the post-tensioning process. Since the bridge was phased, the post-tensioning occurred on different days.
Figure 7.9 – Strains in VW Gages During Post-tensioning. Phase I and II are tensioned on different days. Phase I gages are over Bar #3 and Phase II over Bar #2. The gage on Joint #9 broke loose and did not register.

Most of the gages show about 40-60 microstrain. As shown in Figure 7.7, the 6 inch long gages are attached to the beams and span joint. This means they are reading an average strain for the beam concrete and joint grout. To calculate a stress, a weighted average value of E should be used. The actual E values are not known, but experience shows that good estimates are 3,000,000 psi for grout and 4,000,000 to 4,500,000 for the concrete. Considering that the joint is only 1.75 inches wide at the top, using an value of 4,000,000 for the system is not unreasonable.

Given the measured values of 40-60 microstrain, the stress would be calculated at 160 to 240 psi – approximate 2 to 3 times the expected 72 psi. However, there are several reasons for this difference:

1) The strain is measured at the top. The PT bars are clearly eccentric and the configuration would suggest the development of additional compressive stress in the top.
2) While post-tensioning the bridge, the bridge was warming up due to normal daily temperature rise. This would cause additional compressive stresses.

3) The elastic modulus is not known. If $E = 3,000,000$ psi, the stresses drop to 120 – 180 psi.

The gage on Joint #1 shows a much higher strain. This is not unexpected as this joint is closest to the jacking point. Also, this is the side with the 3 foot wide beam. It is expected that there will be some localized stress here. However, Joints #8 (the construction joint) and #10 in Phase II show very high strains. It appears that the data for Joints 8 and 10 may be due to malfunctioning gages. The gage located on the construction joint between Bars 2 and 3 shows the more usual 50 microstrain.

The strains in the joints do appear to be uniform. As an example, Figure 7.10 shows the strains along Joints #3 in Phase I. The strains along the joint are reasonably uniform. Note that it is possible to see when bars near the gages are tensioned by the sudden change in the gage strain.

After post-tensioning, there is a decrease in strain. This is due to a combination of a decrease in temperature (which will cause a tensile strain) and creep of the concrete and grout.

Figure 7.10 – Strains along Joint #3 – Phase I.
7.4 ENVIRONMENTAL MONITORING

The bridge was completed in December of 2006. In October of 2008, gages were installed on the bridge to monitor environmental effects. Figure 7.11 show the gage positions. Normally, monitoring instruments are fairly safe as they are under the bridge and are not seen. This bridge is over a bicycle path, so it is not uncommon for pedestrians and cyclists to be under the bridge. Some of the instruments were vandalized, notably the deflection measuring devices. Figure 7.11 shows only those instruments which survived and were useable; which were all vibrating wire gages.

![Diagram of vibrating wire gages](image.png)

Figure 7.11 – Vibrating wire gages for environmental and truck load monitoring. All gages are at midspan. Gages on beam monitor longitudinal strain; lateral strain measured at joints.

Figures 7.12 and 7.13 show the strains for the beams and the joints over a period of 2 months (October to December). The vibrating wire gages do not react fast enough to measure traffic induced strains, so the strains shown represent environmental effects. The beams display an interesting behavior. The expected behavior is that the strain should become compressive (negative) when the temperature rises. This is because the structure tries to expand but is restrained. When the temperature decreases, tensile strains should occur. This is exactly what happens in both the beams and joints with the daily temperature fluctuations. In general, the joints display this behavior as the average
temperature changes. However, the beams in Phase I exhibit the opposite behavior. Compressive strains develop as the average temperature drops. Beam 9, which is in Phase II, exhibits the expected behavior. It is possible the odd behavior of the beams in Phase I is caused by phasing, however the exact cause is not clear.

Figure 7.12 – Longitudinal strain in beams
For the beams, the daily fluctuation is, at most, 100 microstrain. This represents a stress of about 400 psi. The seasonal changes are about 150 microstrain, or about 600 psi. Given that these are prestressed beams, these stress levels should not cause problems.

The joints show transverse strains of no more than 100 microstrain in tension. Depending on the assume E value for the joints, this is a stress of 300 – 400 psi. This might have caused cracking if the joints were not post-tensioned, but the presence of the post-tensioning should prevent cracking. Visual inspection of the bridge did not show any signs of cracking or leakage at the joints.

### 7.5 TRUCK LOAD TESTING

In December 2008, two years after the bridge was opened, a truck load test was conducted. A total of four (4), tandem axle trucks, each weighing approximately 50 kips. The trucks were parked on the bridge in six different configurations to measure load distribution. Table 7.1 shows the truck weights and Figure 7.14 shows the truck dimensions.
### Table 7.1 – Truck Weights

<table>
<thead>
<tr>
<th>Truck #</th>
<th>Front Axle (k)</th>
<th>Front Tandem (k)</th>
<th>Back Tandem (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>517</td>
<td>13.65</td>
<td>16.75</td>
<td>16.35</td>
</tr>
<tr>
<td>611</td>
<td>13.65</td>
<td>19.60</td>
<td>18.90</td>
</tr>
<tr>
<td>781</td>
<td>13.45</td>
<td>17.75</td>
<td>16.80</td>
</tr>
<tr>
<td>858</td>
<td>13.15</td>
<td>18.55</td>
<td>18.95</td>
</tr>
</tbody>
</table>

**Figure 7.14 – Test Truck (Test 3 is shown)**
Figure 7.15 – Truck Load Tests. Maximum longitudinal beam strain shown.
Figure 7.15 shows the six truck positions with the maximum longitudinal strain in the beams. Note that the gage on Beam #5 did not read and the gage on Beam #3 was suspect as it gave unrealistically high strain values.

The expected strain values depends on the assumptions made about load distribution. Consider Test #6, which duplicates the AASHTO load case of one truck in each lane. The calculated distribution factor for this bridge is approximately 0.27. Assuming $E = 4000$ ksi, a strain value of 90 microstrain would be expected. If it is simply assumed that all the beams take equal loads, the strain value would be 80 microstrain. However, this ignores the stiffening effect of the concrete railings and sidewalk. If the sidewalk and railings are considered, the expected strain value would be about 40 microstrain.

In Tests 1-4, there appears to be a fairly reasonable strain distribution. If the sidewalk and rail are counted and it is assumed all beams take equal moment, strains of 35 microstrain would be expected. Beam 4 appears to pick up quite a bit more strain than the other beams when trucks are park on it. The reason for this is not clear. Another anomaly is the relatively low strain in Beam 9, which is on the other side of the construction joint. This may be caused by phasing of the construction.

Questions about the effect of phasing arise because of the response to Test #5. In this test, all of the trucks are parked on Phase II. Several of the beams in Phase I, near the construction joint, show negative (compressive) strains. In Test 6, where two of the trucks are parked on the Phase II side of the bridge and one straddles the construction joint, the strains in these beams are very low. Figure 7.16 shows the strains in the gages with the x axis being time, in minutes. Note that during Test 5, Beams 6, 7 and 8 show compressive strains. The strains then go to zero when the trucks leave the bridge. When the trucks are repositioned, the first trucks placed were those on Phase II side of the bridge. Note that all of the Phase I beams now show compressive strains until trucks are placed on the Phase I side, when tensile strains then develop. Clearly, there is something happening at the phase line, but there is not enough data to determine exactly what is happening. However, data from environmental monitoring showed a difference at the phase line. Behavior at the phase line warrants additional investigation.

Figure 7.17 shows the behavior of the joint between Beams 5 and 6. This is fairly typical data for all the joints. Note that maximum strains do not exceed 25 microstrain. Allowing for $E$ values of 3000 to 4000 ksi, the stress is 75 to 100 psi tension. This is well below any values of stress which cause distress in the bridge.

In general, the truck load tests indicate that the bridge is behaving as unit. There appears to be both longitudinal and lateral flexural continuity.
Figure 7.16 – Longitudinal strain in the beams, Tests 5 and 6

Figure 7.17 – Strain in Joint 5 (between Beams 5 and 6)
7.6 FINAL INSPECTION

During the truck load tests, the bridge was given a final inspection. In general, the bridge was excellent shape. The biggest problem with adjacent box girder bridges is leakage through the shear keys and there was no sign of leakage. The bottom of the bridge appeared sound.

The top of the bridge had an integral wearing surface. After grouting the joints, the top surface was simply ground to profile. The shear key joints appeared sound and in good condition. The only distress noted was at the end of the girders. The girders are tied into the approach slab with a rebar and strand connection. There is cracking (limited to the top of the girder) at this connection (Figure 7.18). It cannot be ascertained if this is temperature related cracking or negative moment cracking from some fixity developing at the end of the girders (Figure 7.19).

![Image of bridge connection]

Figure 7.18 – Connection of approach slab to the beam.

![Image of cracked beam]

Figure 7.19 – Cracking at the approach slab/beam joint. Cracks in beams enhanced for clarity.
CHAPTER 8

DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

8.1 DISCUSSION

The objective of this project was to examine whether bridges could be built “Smarter, Faster and Better.” The simple answer is yes, bridges can be built Smarter, Faster and Better.” However, the barriers to doing this are not technical. A previous report to ODOT spelled out ways to build bridges faster (Miller 2002). These methods have been confirmed by the work done on this project. In essence, the method is:

1) Use prefabricated elements.
2) Order necessary materials early and store so they are available when needed.
3) If possible, do as much assembly of these elements as possible before closing the existing bridge.
4) For elements to be assembled on site, use a mock-up to check the fit beforehand.
5) Create streamlined procedures for acceptance of field work.
6) Close the bridge and use a detour rather than using phased construction.

However, the true issue here is the human element. In this research, the barriers to building bridges fast, well and inexpensively were not technical, but were almost totally related to human interactions. The bridges which performed well in terms of short construction times did so because of a general atmosphere of co-operation between the contractor, ODOT personnel and the engineer of record. Contractors in these cases seemed to have a motivation to get the job done, even when the motivation was not financial. Poor performers were generally marked by some type of conflict or some type of error. In general, all of the projects had delays due to “normal” occurrences like weather or equipment breakdown. The better performing projects had contractors who were better able to deal with these difficulties.

Salem, Miller and Despande (2008) looked at fast-track projects. The bridges studied in the SI-9 project would not be considered fast-track, but accelerated. The difference between the two is that accelerated construction follows a traditional method. An accelerated project is completely designed and then constructed. The construction stage is compressed, but there is no overlap between design and construction. In fast-track, construction begins before the design is complete. The design may be as little as 20% complete when construction begins; although 40-50% complete is more normal. While accelerated and fast-track are not the same process, they have enough similarities that principal which apply to fast-track also apply to accelerated construction.

Salem, et al. (2008) used a survey of 30 fast-track projects as well as surveys of industry experts to identify the Construction Industry Institute Best Practices which led to successful design in fast-track. These were:
1) Pre-project planning. Many respondents to the surveys indicated that this was the one area they wished they had spent more time on. The survey results showed the more time spent on pre-project planning and the more carefully the planning was done, the greater the chance of success.

2) Constructability. According to Salem, et al. “Constructability can be defined as the effective and timely integration of construction knowledge into the conceptual planning, design, construction and field operations of a project to achieve the overall project objectives in the best possible time and accuracy at the most cost-effective levels.”

3) Alignment. Alignment is when the efforts of all of the participants are aligned to accomplishing the same goal. This might seem obvious, but it isn’t. Consider a typical bridge project. The field engineer might have a goal of getting the bridge done quickly to avoid complaints from the local motorists. The District might be concerned with keeping the project in budget, regardless of the time factor. The contractor might be working on several projects at once and may be more concerned with using his resources most efficiently without regard to how it would affect the time schedule of a single project. Here, the participants are not all aligned to goal of getting the project done quickly.

4) Partnering. Partnering is when the participants are working together as team to achieve the same goal. It is different from alignment in that alignment is about getting everyone “moving in the same direction.” Partnering is about establishing a relationship of trust.

5) Change Management. A critical requirement for fast-track is the ability to manage the inevitable changes and modifications to the original design. In a traditional project, change orders can take days or weeks to approve. Fast track projects condense this process. Use of Building Information Systems (BIM) greatly improves and simplifies the change management.

6) Work force issues. One key to successful fast track projects is having experienced personnel; people with an institutional memory who remember past mistakes and take active steps to avoid them. Another key is having a low turnover of personnel. Constant changes in the workforce disrupt the project.

7) Procurement. Most fast-track projects use trusted, sole source suppliers and standardized equipment. Use of standardized equipment simplifies the design process. Use of trusted suppliers guarantees the availability of materials/equipment. It was found that fast track projects will often purchase large quantities of “stock” material (e.g. rebar) in advance to assure the material is available.

Note that all of the practices listed could be applied, in some way, to the bridges studied in the SI-9 project.

The most successful project was PIC-22. The bridge was completed 10 days ahead of schedule and at 65% of the original estimate (including the contractor’s incentive). Data collected after the project showed were many of the methodologies and best practices were employed.
PIC-22 was a design-build project. Information from the post-construction meeting showed that the engineer and the contractor spent a tremendous amount of time on pre-project planning. Fabrication and supply line issues were solved before the project started. It was clear that the project had good alignment. Everyone involved knew that failing to open the bridge on time would have a negative economic impact on the city, so everyone was aligned to the goal of finishing on time. It was noted that contractor’s and ODOT’s personnel sometimes “slept in the trailer” to be available to make key decisions (Alignment and change management).

The engineer considered the constructability issues. Prefabricated elements were used and existing foundation were re-used. The bridge was designed as a continuous for live load steel structure; common for concrete but very unusual for steel. This was done to speed up the erection process. Stay-in-place steel forms were used cut deck forming time. In this case, the contractor set up the beams off site, but with proper relative elevations, so that the “ladders” which hold the SIP forms could be prefabricated.

One key element was that the incentive was large, $500,000. The contractor noted that this large of an incentive was enough to justify bringing in the best and most experienced employees from out of town.

Project GUE-513 provides an excellent example of where application of these principles led to success and where failure to implement these principles let to problems. Prior to the start of project, there were numerous meetings between the contractor, the engineer and ODOT personnel. During this time, potential problems were identified and solved before construction started. There are two good examples of how this pre-project planning worked. When post-tensioning, ODOT rules required that one bar or strand group be tensioned and the results sent to Columbus for approval before any more tensioning occurred. For GUE-513, ODOT provided a method of immediate field approval. In the second case, the contractor wanted to cut the slab loose from the abutments before closing the bridge. Had he attempted to do this, the ODOT field engineers would have stopped the project until they were assured this procedure was safe. By identifying this as a problem beforehand, the necessary safety checks were done and approvals were secured before construction started.

During the construction phase, the contractor and the ODOT field engineer held frequent partnering meetings. This helped to resolve many problems without delaying the project. The ODOT field engineer was very experienced and was given wider than normal latitude to make decisions. This contributed to the success of the project.

GUE-513 did have some problems; largely pre-project planning and work force experience issues. The engineer specified a certain type of fast setting grout. Although it was clearly spelled out in the bid documents, in retrospect, the grout should have somehow been red flagged for the contractor. The contractor did not realize this special grout was needed and he did not have it available when needed. The grout situation was exacerbated when the contractor was unable to find a grout which was available, met the engineers’ specification and was on the ODOT approved list. In the end, a compromise
grout was used. However, the grouting contractor was not familiar with this grout and had problems installing it.

Another problem occurred with precast elements. The engineer had thought ahead and required a mock-up assembly to check fit and alignment. However, no one remembered the bridge had a crown. The precast deck was fit-up without a crown and was then misaligned in the field. It should be noted that ODOT personnel had an institutional memory and this mistake was not repeated on subsequent projects (MOT-70, FAI-22).

CLI-730 was the least successful of the projects. The specific problems are detailed in Chapter 5. This was one of the first laterally post-tensioned bridges and many of the problems can be attributed to a lack of experience. However, there were other shortcomings. The contractor noted that a pre-bid meeting would have helped, they were unaware this was a pilot project until the preconstruction meeting. Although there were frequent meetings between ODOT and the contractor, the process was not true partnering. Part of this was a reluctance by ODOT personnel to “specify means and methods,” because, traditionally, ODOT personnel do not do that. However, in the post construction meeting the contractor made several comments which indicated that there were times when a little guidance would have helped. It should be noted that the “lessons learned” on CLI-730 were applied to the adjacent box girder bridges MOT-70 and FAI-22 and the deck panels on HAN 75, making these projects successful.

In all, when the principles listed at the beginning of this section were applied, the project was successful. It cannot be overstated that difference between successful projects and less successful project is not whether or not problems occurred; all of the projects had problems. The difference between success and lack of success is having effective pre-project planning and having systems in place to effectively deal with the problems when they occur.

8.2 GENERAL CONCLUSIONS

1) The barriers to building bridges faster, better and more efficiently are not technological, but are largely human factors. Bridges can be built faster, better and more efficiently when the following principles are applied:
   a. Effective pre-project planning.
   b. Designing the project for fast and efficient construction.
   c. Aligning all project personnel to the same goal.
   d. Creating and maintaining an effective partnering strategy.
   e. Creating and maintaining an effective change management system.
   f. Use of an experienced, low turnover workforce.
2) From a technological standpoint, bridge construction can be accelerated by:
   a. Use prefabricated elements.
   b. Order necessary materials early and store so they are available when needed.
c. If possible, do as much assembly of these elements as possible before closing the existing bridge.
d. For elements to be assembled on site, use a mock-up to check the fit beforehand.
e. Create streamlined procedures for acceptance of field work.
f. Close the bridge and use a detour rather and using phased construction.

3) Incentives and disincentives have only a marginal effect, if any at all, on time to completion.
a. Incentives: Incentives are only effective if they cause the contractor to fundamentally alter the construction process (e.g. work overtime, add second shifts, use innovative construction techniques and materials). Data from benchmarking and post-construction evaluations show that only the largest jobs have enough incentive to cause this change in the construction process. For a typical job, the incentives are usually not large enough. This does not mean that the incentives should be abandoned. Data suggest that, while the contractor may not make large changes to collect the incentive, the presence of the incentive might cause some small changes in behavior.
b. Disincentives/penalties: Benchmarking data suggest that these penalties are rarely enforced. The presence of penalties may prevent egregious violations of the contract, but in most cases the contractors can justify the delays well enough to avoid penalty.
c. There was some suggestion in post-construction meetings that contractors include incentives/disincentives in the bid. If the contractor feels the job requirements are unrealistic, they will increase their bid to account for the risk of a penalty. If they feel they have a good chance of getting the incentive, they may decrease the bid to win the job and use the incentive to make up the difference.
d. Incentives/disincentives are not usually passed on subcontractors. This can create a problem as the subcontractor has no reason to worry about the schedule.

4) The precast concrete deck panels used on HAN-75 preformed very well. The use of precast deck panels did not increase the speed of construction on this project. ODOT District personnel thought that use of the panels had the potential to speed up construction if a better construction methodology were found.

8.3 SPECIFIC CONCLUSIONS AND RECOMMENDATIONS FOR ADJACENT BOX GIRDER BRIDGES

1) Use of a full depth shear key is recommended. This recommendation is based on the work of Dong (2002) and the recommendation of Hanna, Morcous and Tadros (2009).
2) The Commentary to AASHTO LRFD Specifications (C4.6.2.2) recommends that lateral post-tensioning in an adjacent box girder bridge provide a stress of 250 psi. However, this value has never been adequately defined (250 psi on the entire side of the girder? 250 psi on the shear key? 250 psi on the
nor has it been adequately justified. Hanna, Morcous and Tadros (2009) provide more justifiable values for lateral post-tensioning. The lateral post-tensioning of FAI-22, MOT-70 and CLI 530 did not meet the AASHTO recommendations, but were well within the Hanna, et al. recommendations.

3) FAI-22 was laterally post-tensioned in two phases. In the first phase, 8 girders with a combined width of 31 ft. 7 in. were tensioned together. In phase two, 9 girders with a combined width of 36 ft. 9 in. were tensioned together. There was concern that when tensioning this many girders together, the shear keys in the middle would not be compressed. The data shows that these joints did compress and that the compression was fairly uniform from joint to joint.

4) FAI-22 was post-tensioned at six discreet diaphragms. There was concern that the joints would not receive sufficient compression between the diaphragms. Again, the data show that joints compressed in a fairly uniform manner along the length of the joint.

5) An attempt was made to place load cells between the girders to monitor both applied load and loss of post-tensioning. Unfortunately, these did not work.

6) Subsequent load testing showed that the bridge had adequate lateral flexural continuity, as required by the AASHTO LRFD Specifications, Article 4.6.2.2.

8.4 IMPLEMENTATION RECOMMENDATIONS
1) To judge the effectiveness of bridge projects, ODOT needs to develop a benchmarking system. It is recommended that:
   a. ODOT form a benchmarking team to determine what data is needed and a collection method.
   b. There needs to be a consistent way to define “where items go.” For example, under the current system, an approach slab might be part of the bridge in one project and part of the pavement in another.
   c. Create an improved and consistent format for construction diaries. Frequently, the causes and durations of delays and important dates (like date of bridge closing) are missing. This information is necessary for benchmarking.
   d. Develop a consistent method for tracking contractor progress.

2) ODOT currently has a partnering program. To improve this program and to add alignment, it is recommended that everyone involved with the project have training and information on partnering. There should be formal reviews of the partnering process on each project to be sure it being done properly.

3) ODOT currently reviews plans for constructability, but every effort should be made to be sure that this is done and done properly as constructability reviews are extremely important. ODOT should review its current training in this area to be sure it is adequate.

4) It is recommended that formal pre-project planning strategies be adopted. ODOT should consider:
   a. Involving the engineer of record in the planning process so that the design reflects not only good, technical engineering, but also good planning for fast and effective construction.
b. Use of the pre-bid meeting. This is essential to flag out any odd or unusual details, materials and processes. Pre-bid meetings can provide feedback which might improve the planning process.

c. Use of preconstruction meetings for planning. Currently, preconstruction meetings tend to be informational. The successful projects in this research used the pre-construction meetings for planning.

5) ODOT may wish review its change management system. Fast and effective change management is a key factor in project success.

6) Data suggest that the presence of incentive and penalties have little effect on performance, except for the largest jobs. ODOT may wish to evaluate the effectiveness of this program.

7) ODOT should consider working with the State Highway Patrol on permit issues. In two cases (MOT-70 and PIC-22), shipments of vital components were delayed because of permitting issues. In both cases, the problem was a “technical” violation not a safety issue; e.g. in one case a truck broke down and the existing permit could not be used with the replacement truck. The first priority should be keeping Ohio’s roads safe, but some provision should be made to allow shipment of vital components if the violation is not a safety issue.

8) In every project, the use of grout was a problem. It is recommended that ODOT review the grouting materials list and grouting standards. Specifically, the review should consider:

   a. The development of a “ready-mixed” grout as an alternative to bagged material.

   b. Creating a procedure to allow the use of special purpose grouts which are not on the currently approved list.

   c. Creation of detail for grouting full depth shear keys in adjacent box girder bridges.

   d. Examining the property variability and problems with grout installation. Perhaps a change in specifications or procedures is needed.

8.5 RECOMMENDATION FOR FUTURE RESEARCH ON BOX GIRDERS

The results of this study provided valuable insight on laterally post-tensioned adjacent box girder bridges. However, it was not possible to answer all of the questions about the adequacy of the lateral post-tension. This was largely because of the limitations of trying to measure an in-service bridge. For proper instrumentation, the instrumentation must be designed into the bridge. Often, the bridge is already designed by the time the decision to instrument it is made. Proper instrumentation also requires things like leaving holes in the structure, placing load cells under bearing plates and adding ducts for wires. All of these have potentially negative impacts on bridge durability and may not be practical for in-service bridges. Proper measurement requires exacting control of environmental and load conditions; impossible with an in-service bridge. Finally, the contractor for an in-service bridge is trying to get the bridge done and tasks of the research team interfere with this goal. Co-ordination with contractors is often difficult and contractors will sometimes remove or damage instrumentation.
It is recommended that:

1) A full size, experimental bridge should be built. Since ODOT phases construction, only one phase of 7 – 8 beams would be needed.

2) The bridge should use full depth shear keys and the lateral post tensioning be consistent with the recommendations of Hanna et al. or the ODOT standard, whichever is greater.

3) Instrumentation should be “designed into the bridge.” Since this is not an in-service bridge, durability will not be a concern so the emphasis can be on proper instrumentation. The instrumentation should include load cells under the bearing plates of the PT bars, strain gages in the top and bottom flanges of the girders, strain gages in the shear keys and load cells between the girders which would bear on the surfaces of adjacent girders.

4) The bridge should be built in an ODOT yard where a temporary shelter can be erected. The shelter can be temperature controlled to eliminate environmental concerns and then later removed to measure environmental effects.

5) If desired, the test structure can be load tested either with hydraulics or dead weights.
References


Dong, X., Temperature and Traffic Forces on Shear Key Connections for Adjacent Box Girder Bridges, a dissertation in partial fulfillment of the requirements for the degree of Doctor of Philosophy, University of Cincinnati, 2002.


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