NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP

Web-Only Document 243:

Recommended Guidelines for Prefabricated Bridge Elements and Systems Tolerances and Recommended Guidelines for Dynamic Effects for Bridge Systems

Michael P. Culmo
CME Associates, Inc.
East Hartford, CT

In Association with
Marvin W. Halling
Marc Maguire
Utah State University
Logan, UT

Dennis Mertz

Contractor's Final Report for NCHRP Project 12-98
Submitted December 2017

The National Academies of
SCIENCES · ENGINEERING · MEDICINE

TRB
TRANSPORTATION RESEARCH BOARD

NCHRP

Web-Only Document 243:

Recommended Guidelines for Prefabricated Bridge Elements and Systems Tolerances and Recommended Guidelines for Dynamic Effects for Bridge Systems

Michael P. Culmo
CME Associates, Inc.
East Hartford, CT

In Association with
Marvin W. Halling
Marc Maguire
Utah State University
Logan, UT

Dennis Mertz

Contractor's Final Report for NCHRP Project 12-98
Submitted December 2017

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Academies of Sciences, Engineering, and Medicine.

The National Academies of
SCIENCES · ENGINEERING · MEDICINE

TRB
TRANSPORTATION RESEARCH BOARD

The National Academies of

SCIENCES • ENGINEERING • MEDICINE

The **National Academy of Sciences** was established in 1863 by an Act of Congress, signed by President Lincoln, as a private, non-governmental institution to advise the nation on issues related to science and technology. Members are elected by their peers for outstanding contributions to research. Dr. Marcia McNutt is president.

The **National Academy of Engineering** was established in 1964 under the charter of the National Academy of Sciences to bring the practices of engineering to advising the nation. Members are elected by their peers for extraordinary contributions to engineering. Dr. C. D. Mote, Jr., is president.

The **National Academy of Medicine** (formerly the Institute of Medicine) was established in 1970 under the charter of the National Academy of Sciences to advise the nation on medical and health issues. Members are elected by their peers for distinguished contributions to medicine and health. Dr. Victor J. Dzau is president.

The three Academies work together as the **National Academies of Sciences, Engineering, and Medicine** to provide independent, objective analysis and advice to the nation and conduct other activities to solve complex problems and inform public policy decisions. The National Academies also encourage education and research, recognize outstanding contributions to knowledge, and increase public understanding in matters of science, engineering, and medicine.

Learn more about the National Academies of Sciences, Engineering, and Medicine at **www.national-academies.org**.

The **Transportation Research Board** is one of seven major programs of the National Academies of Sciences, Engineering, and Medicine. The mission of the Transportation Research Board is to increase the benefits that transportation contributes to society by providing leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal. The Board's varied committees, task forces, and panels annually engage about 7,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation.

Learn more about the Transportation Research Board at **www.TRB.org**.

Table of Contents

List of Figures and Tables

Author Acknowledgments

The research reported herein was performed under NCHRP Project 12-98 by CME Associates, Inc. (CME). Michael P. Culmo, P.E., Chief Technology Officer at CME was the Principal Investigator. The Co-Investigators primarily responsible for the dynamic portions of the work were Dr. Marvin W. Halling and Dr. Marc Maquire of Utah State University. Further assistance was given by Dr. Paul Barr of Utah State University, Kris Johnson and Sattar Dorafshan, both graduate research assistants at Utah State University, and Jenifer Pixley, a Bridge Design Engineer at CME Associates, Inc.

The authors also acknowledge the contribution of Dr. Dennis Mertz during the development of this work.

Abstract

This report documents the results of 2 studies. The first study led to the development of guidelines for tolerances for Prefabricated Bridge Elements and Systems (PBES). The tolerance guidelines were developed based on a synthesis of current industry practice. Statistical methods were developed to analyze fabrication data for future specification development. A statistical method was developed to account for multiple tolerances factoring into joint width and connection tolerances between prefabricated elements. Monte Carlo simulations were completed in order to establish probability adjustment factors for these tolerances. To complete the work, a tolerance guideline was developed in standard AASHTO format.

The second study involved Dynamic Effects of Accelerated Bridge Construction (ABC) Bridge Systems. These include dynamic friction effects of lateral bridge slides and dynamic effects for Self-Propelled Modular Transporters (SPMT) bridge moves. Various common sliding systems were testing in the laboratory, which led to the development of recommended specifications for design and construction of these systems. Full scale testing was completed for SPMT bridge moves. The hypothesis for the research was to treat an SPMT move as a man-made earthquake. Upper bound dynamic effects were measured on an actual SPMT with various levels of load applied. Based on the results, a dynamic response spectrum was developed for horizontal and vertical dynamics. A design method was then developed, which was included in a guideline document written in standard AASHTO format.

Summary

ABC has become commonplace throughout the United States. Many projects have been completed by virtually every state. Many ABC technologies have been researched and developed; however there are no specifications or guidelines for tolerances of prefabricated bridge elements or dynamic effects for construction and placement of bridge systems. Designers are currently designing ABC projects using engineering judgement and the *AASHTO LRFD Bridge Design* and *Construction Specifications.*

The main deliverable in this project is 2 guideline documents written in AASHTO format. The development of the specification was based on a literature search for each technology, laboratory research for friction of sliding systems and field research for dynamic response of SPMTs. The work was reinforced by the actual experience of the project team members who have built ABC projects. The project team actively engaged the AASHTO T-4 Construction Committee of the SCOBS and the Florida International University ABC University Transportation Center.

One of the major goals of any research is implementation. The project team has a close relationship with the AASHTO T-4 Construction Committee through our work on NCHRP Project 12-102. The team plans on working with this committee on the implementation of the 2 guidelines that were developed under this project. The team would ultimately like to merge these documents with the recently adopted (but yet to be published) AASHTO ABC Guide Specifications.

CHAPTER 1

Background

1.1 Problem Statement and Research Objectives

ABC using prefabrication is not new to the United States Bridge market; however the use of ABC technologies and prefabrication in the United States has expanded dramatically over the last decade. Prior to this period, the use of ABC was limited to special projects that necessitated the use of ABC because no other reasonable alternative existed. At this time, agencies have discovered that ABC produces other benefits, some of which include:

- Reduced Onsite Construction Time (Prefabrication Off site)
- Reduced Mobility Impacts
- Reduced Environmental Impact Time
- Reduced Road User Costs
- Improved Safety (Construction Crews and Road Users)
- Improved Quality (Plant Produced Prefabricated Products)

ABC projects make use of "PBES". This title covers 2 major types of projects that are in use today. The first is prefabricated bridge elements. FHWA defines prefabricated bridge elements as:

> *Prefabricated elements are a category of PBES which comprise a single structural component of a bridge. Under the context of ABC, prefabricated elements reduce or eliminate the onsite construction time that is needed to build a similar structural component using conventional construction methods. An element is typically built in a prefabricated and repeatable manner to offset costs. Because the elements are built under controlled environmental conditions, the influence of weather related impacts can be eliminated and improvements in product quality and long-term durability can be better achieved.*

The second type of project is referred to as prefabricated bridge systems. FHWA defines prefabricated bridge systems as:

> *Prefabricated Systems are a category of PBES that consists of an entire superstructure, an entire superstructure and substructure, or a total bridge that is procured in a modular manner such that traffic operations can be allowed to resume after placement. Prefabricated systems are rolled, launched, slid, lifted, or otherwise transported into place, having the deck and preferably the parapets in place such that no separate construction phase is required after placement. Due to the manner in which they are installed, prefabricated systems often require innovations in planning, engineering design, high-performance materials, and "Structural Placement Methods".*

Project 12-98 looks to cover several outstanding issues related to each of these methods of construction. Numerous agencies, Universities, and research institutions (including NCHRP) have

stepped up and completed significant amounts of research that are relevant to ABC relating to design, performance and construction of PBES. To date, there has been little research related to element tolerances and the dynamics associated with installing bridge systems.

Most bridge design engineers are not well versed in the field of erection and fabrication tolerances. This is due to the fact that most bridges are built using cast-in-place concrete. The limits of understanding of tolerances are typically limited to camber in beams. Bridge engineers understand that beam camber tolerances are accounted for in the deck "haunch" or "blocking gap" that represents a gap between the top of the fabricated beam and the underside of the bridge deck. In order to assemble multiple prefabricated bridge elements, similar "gaps" need to be designed in order to accommodate the fabrication tolerances of the elements.

To date, a significant number of problems with prefabricated bridge construction projects can be attributed to issues with tolerances. In 2012, a paper was presented at the TRB annual meeting entitled "First Fully Prefabricated Full-Depth Deck Panel Bridge System in Michigan: Challenges and Lessons Learned". This paper, authored by Professor Upul Attanayake (et al) from Western Michigan University noted problems with fit-up of precast concrete full-depth deck panels. This paper underscores the need to accommodate fabrication and construction tolerances for prefabricated elements. A guide specification that includes standardized element fabrication and construction tolerances could prevent many of the issues that have been seen early prefabricated bridge element projects.

The concept of prefabricating an entire bridge superstructure off site and moving it into its final position is foreign to most bridge design engineers. Through the hard work of the FHWA, AASHTO, and several key state Departments of Transportation such as Florida and Utah, the concept of moving an entire bridge superstructure has become more common in the United States. Early uses of bridge system installations were specialized projects that relied heavily on the expertise of the heavy lift sub-contractor in order to provide a successful project. With the increased use of bridge system moves, more heavy lift companies are entering the market. Not all companies are equally versed in the intricacies of bridge moves, which bring about the need to have consistent provisions governing the design and construction of a bridge system.

Many agencies have also expressed a resistance to the widespread use of ABC citing the lack of AASHTO specifications that should be used for the design and construction of these bridges. The lack of consistent specifications and scattering of technologies is hampering the more widespread use of ABC. Increased use of ABC through better specifications and guidelines will not only benefit the bridge design and construction community, but the greater transportation community as a whole.

1.1.1 Research Objectives

There are 2 objectives to this research project. The first is to establish guidelines for fabrication and erection tolerances of PBES. This would include tolerance criteria for project specifications that could be used for acceptance of fabricated elements, and information on how a designer should detail a prefabricated bridge using the specified tolerances as the basis. The end deliverable is a guideline document, written in the format of the AASHTO LRFD Bridge Construction Specifications. A future goal would be to obtain approval of the document from the AASHTO Subcommittee on Bridges and Structures (SCOBS), which would then publish the document as a national guideline.

The second objective of this project is to determine the maximum anticipated dynamic effects that are imposed on the bridge system and the supporting temporary works during bridge system installations.

The research covers the most common form of bridge systems including lateral sliding and installations using SPMTs. The end deliverable is a guideline document, written in the format of the AASHTO LRFD Bridge Construction Specifications. Dynamic effects during bridge system installations have an impact on both the bridge structure and the supporting temporary works. Based on this, some of the provisions of the guideline will correspond to the AASHTO Guide Specification for Bridge Temporary Works. The 2 guidelines provide practical, implementable and timely solutions to cover the issues noted above.

1.1.2 Terminology and Definitions

The AASHTO Technical Committee Number T-4 of the Subcommittee on Bridges and Structures has established standard terminology and definitions for ABC. These terms have been used throughout the development of the proposed Guide Specifications. The terms and definitions, which are not subject to changes, are included in the proposed guideline documents.

1.1.3 Related Research Projects

1.1.3.1 NCHRP Project 12-102

A second NCHRP ABC related research project entitled "Recommended AASHTO Guide Specification for ABC Design and Construction" (NCHRP 12-102) was underway during the development of this project. The objective of that project was to develop AASHTO Guide Specifications for ABC Design and Construction. That guide specification was completed in 2017 and adopted by AASHTO at the 2017 meeting of the Subcommittee on Bridges and Structures.

Fortunately, the same Principal Investigator was selected for both projects, which facilitated the development of the all of the documents in the 2 projects. The team carefully separated the 2 project specifications in an attempt to not duplicate provisions, which could lead to inconsistencies. There are cases where cross referencing is used to make all of the documents work together.

At some point in the future, the AASHTO Technical Committee (T-4) for Construction may choose to merge all of the documents created in the 2 projects. The potential merging of these documents is beyond the scope of this project. In the interim, the cross referencing should suffice for application of these technologies in ABC projects.

CHAPTER 2

Project Approach

The approach to this project was twofold. One approach was developed for tolerance guidelines, and another approach was employed for dynamic effects. The following sections outline the approach for each portion of the research effort.

2.1 Tolerances

The importance of tolerances cannot be overstated, especially on an accelerated construction project. The ramifications of misfit elements can lead to delays in construction, time overruns and even litigation. The goal of ABC is to minimize the impact of bridge construction on the traveling public. Failure to complete a project on time can result in loss of confidence from the public, which could undermine the significant efforts to encourage ABC that have been completed to date.

Design engineers for bridges often do not consider tolerances during the design and detailing phase of a bridge project. Conventional bridge construction practices make use of cast-in-place concrete for large portions of the structure. Cast-in-place concrete is an infinitely adjustable material. If portions of the structure are built slightly different than the dimensions shown on the plans, the subsequent portion can be corrected by adjusting the formwork of the next pour. The one major type of tolerance that most bridge designers consider is the camber tolerance of the beam elements (both steel and concrete). Variations in beam camber can lead to uneven deck surfaces. For this reason, designers typically detail a gap between the top of the beam and the underside of the deck (often referred to as the deck haunch or blocking gap). Many agencies have standard haunch depths based on past experience; therefore the design engineer need not consider the ramifications of beam camber. In fact, many design engineers are not even aware of the specification limits on camber variation.

Building a bridge with prefabricated elements is essentially the same problem as the deck haunch problem. All fabricated elements are manufactured to a tolerance. Nothing is perfect (in spite of what our CAD systems tell us). In order to ensure that elements fit together in the geometry that we specify, a system of adjustable features needs to be incorporated into the design and details. The approach is similar to the deck haunch details. A series of joints needs to be specified on the contract drawings. The width of the joints is directly related to the specified tolerances. If a designer asks: What is the width of the joint between the elements? The answer is another question: What are the specified fabrication and erection tolerances? Knowing these values, a designer can mathematically calculate the width of the joint. Simply stated, larger tolerances equate to larger joints.

There are 2 major types of tolerances that need to be accommodated in the detailing of a prefabricated bridge. The first is element fabrication tolerance. This includes dimensional tolerances such as width, length, thickness and squareness. The second tolerance type that should be considered is the tolerance on the setting position of the element during construction, also called erection tolerance. This is the variation in the 3-dimensional location of the element after placement.

The team had hoped to accumulate significant amounts of data from actual prefabricated elements. Unfortunately, this data was not available and the gathering of data was not included in the project scope. The team did develop a statistical approach to calculating the probability of tolerances and ways to set appropriate tolerance values given a set of actual fabrication dimensional data. The team did coordinate with industry partners such as the Precast Prestressed Concrete Institute to obtain acceptable tolerance specifications. The team also queried industry partners via an internet poll for the same reason.

Another important aspect of tolerance detailing is the use of working lines and working points. It is critically important to measure element erection tolerances based on a common datum. Setting elements based on center-to-center spacing inevitably results in a loss of overall structure geometry. Figure 2.1-1 shows a detail from the Prestressed Concrete Institute (PCI) Northeast Bridge Technical Committee. All of the horizontal dimensions are measured off a common working line (left column in this case).

The research under this project investigated the probability of element tolerances, with the goal of potentially reducing element joint widths. A typical joint width is affected by 4 or more tolerances. The probability of the maximum of all 4 tolerances being present at one time needs was investigated. The fact is that some of the tolerances will be under width and some will be over width. This research focuses on the probability of multiple tolerance occurrences through the use of Monte Carlo simulations. This mathematical process accounts for occurrence of multiple variables, each with a probability of occurrence. The result is a probability of occurrence of the combination of tolerances.

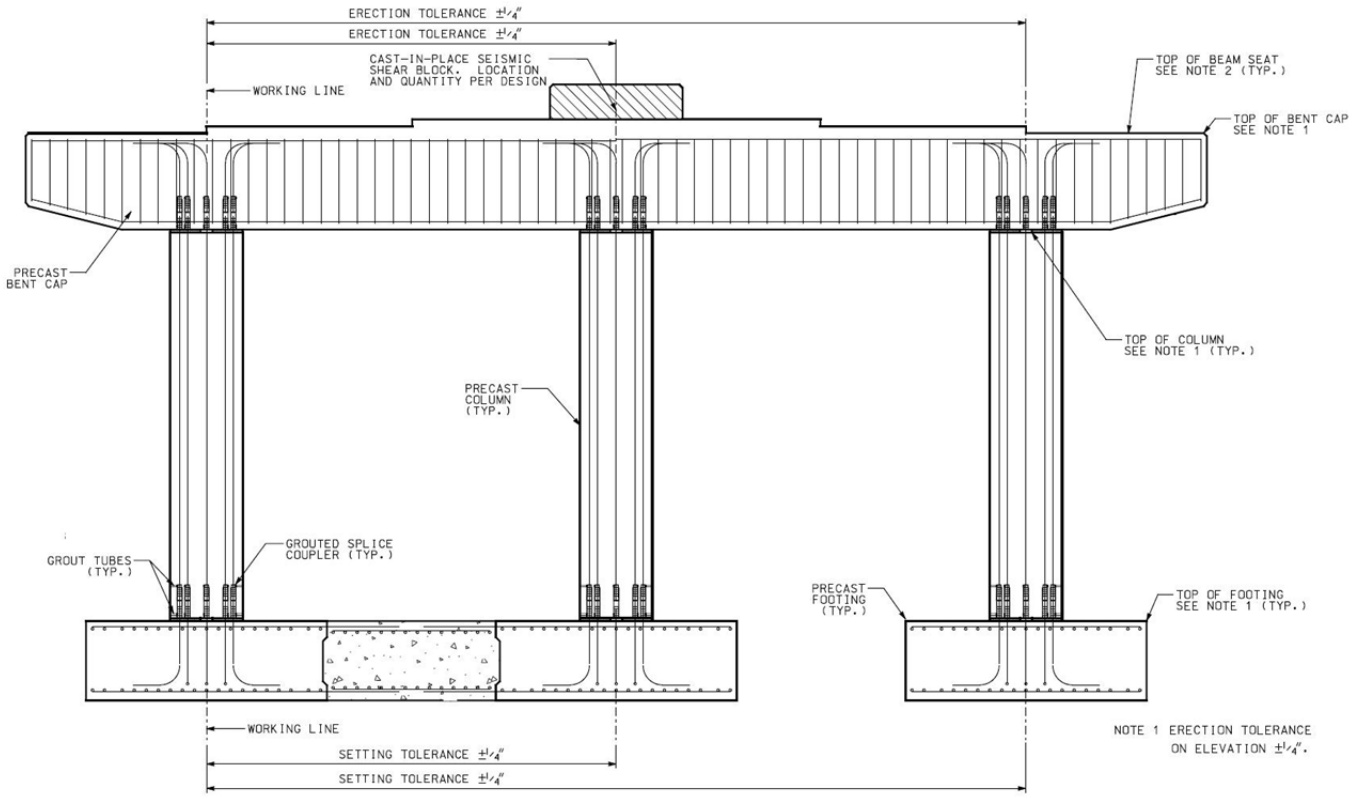


Figure 2.1-1 Example of Element Erection Tolerance Details (source: PCI Northeast)

2.2 Dynamic Effects of Bridge System Moves

There have been over 100 bridge moves completed in the United States in the last 10 years. These projects relied heavily on the expertise of the heavy lift company to determine the dynamic effects on the bridge system and temporary works during the actual move. A number of bridges have been moved with strain gages installed on the bridge members in order to measure strain and stress. These gages have measured stresses induced by vertical "bouncing" of the bridge during the move. There has been little or no investigation of the horizontal dynamic forces imparted on the structure during the moves. Horizontal forces have little effect on most bridge structures; however they can have a dramatic effect on the temporary falsework supporting the bridge. Horizontal dynamic effects are induced into the system during start-ups and stops of the transporters. The magnitude of the lateral dynamic forces estimated by heavy lift companies that are used for the falsework design is typically based on rule-of-thumb experience. With more and more companies entering the market, the level of experience may be reduced. The approach for this research is to formally determine the lateral dynamic effects through full scale testing, and set specific design criteria for the temporary works. This set criterion will dramatically improve safety for all heavy lift contractors by establishing the same standard regarding these horizontal forces.

Field instrumentation of an actual bridge move might seem like an appropriate method for gathering data for this research. Two members of the project team, CME Associates and Utah State University have completed a study of field data from a number of bridges installed in Utah with SPMTs (Rosvall, et al, 2010). The data was gathered by the contractor prior to the start of the study; therefore there was no opportunity for controlling the type of data that was gathered. Still, the study was somewhat useful and resulted in preliminary recommendations for incorporation of dynamic effect specifications into the Utah SPMT manual.

Field data of actual bridge moves is valuable, but only to a degree. On most projects, operators take great care to start and stop as slow as possible, which most likely represents a lower bound on the potential forces. Design specification needs to be based on a specific limit state. It is not safe to take an actual bridge system and test for worst case scenarios. Based on this, the project team measured accelerations on an SPMT loaded with simple weights. Under this scenario, the transporter can be started and stopped at maximum speed to determine the upper bound on design loadings. Another reason to not use an actual bridge is to limit the secondary dynamic effects of the vibration of the falsework and bridge system. The goal of the research is to determine the "base" dynamics, similar to seismic design.

The stiffness of the bridge and falsework also needs to be accounted for in the design process. The approach for this is to develop a response spectrum curve for both vertical and horizontal dynamics. These curves, which can be similar to seismic design, plot dynamic response coefficients against the period of the structure being moved. Through this approach, structures of varying stiffness will generate different forces, thereby accounting for structure and falsework flexibility in the design process.

Lateral sliding has become a very popular method for installing bridge systems. One of the benefits of lateral sliding is in the safety of the system since the bridge system is moved on a fixed (immovable) framework. This quasi-static process essentially eliminates the dynamic effects on the bridge system and the falsework. The one issue that needs investigation is the friction that develops in the system during the move. Various methods of sliding are in use today including rolling devices, Teflon pads (lubricated and unlubricated), and lubricated steel on steel plates. The friction between the bridge and the falsework can be significant. Depending on the configuration of the system, these forces may be imparted to, and accommodated by the bridge structure.

Currently, there is little or no guidance on the amount of friction that typical systems develop. As with SPMT dynamics, most bridge moves are designed based on rules of thumb and past experience. Normally this calculation is in the hands of the heavy lift contractor; however design engineers are called upon to review these calculations; therefore there is a need to have an understanding of realistic friction values. The approach to solving this problem is to simply test various materials in a simple laboratory testing apparatus.

The testing should cover both breakaway friction and dynamic friction. Various common lubricants should be applied to the testing. It is common for contractors to use simple dish washing soap. This low cost option is acceptable for a short-term system such as a lateral move. There are other commonly available lubricants such as bearing grease and oils that may offer better performance. These materials were also studied.

CHAPTER 3

Literature Review and Industry Questionnaire

3.1 Literature Review

The literature review was conducted to identify applicable research as well as owner and industry experiences. This project aims to synthesize previous work while improving design and construction. Current research by universities and state agencies in the area of ABC is extensive; however, work on tolerances and dynamics is limited.

Two main approaches were used for the literature searches. The TRB TRID database was searched. The team also visited every state DOT website to search for local research in the hopes that something was studied and not posted in the TRID database. The following sections contain the results of our searches.

3.1.1 Tolerances

Very little research has been undertaken in the field of precast tolerances; however, several articles were uncovered, primarily from the Precast/PCI. The most important document found is the PCI *Tolerance Manual for Precast and Prestressed Concrete Construction - MNL-135-00* (3). Michael Culmo of CME visited with the Tolerance Committee of PCI during the 2014 and 2016 PCI National Convention. He learned that current PCI tolerances are based on years of experience by fabricators and not based on statistical data. The American Concrete Institute (ACI) also publishes a manual on tolerances entitled *Specification for Tolerances for Precast Concrete, ACI ITG-7-09* (12). This manual is similar to the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction*.

Several journal articles related to tolerances were found (Beerbower (1964), Vander Wal and Walker (1976), Schneider, et al (1993), Lanier, Et al (1985), and Gleich, Et al (2007)) pertained to precast element tolerances and outline the work of the committee. The PCI Tolerance Committee has compiled this knowledge base on tolerances into the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (MNL-135-00)*. This manual is used across the country in vertical construction to specify and manage tolerances for precast products. It is generally not used in bridge construction due to the past practice of only prefabricating bridge girders. The one main bridge tolerance that is of note is the tolerance of camber on prestressed concrete bridge girders. Camber tolerance is a topic that is under significant study by PCI at this time. The authors of this project consider this issue one that is beyond the field of ABC, and applicable to all bridge construction, therefore it was not covered in this project.

The Precast PCI maintains a very organized and robust committee system. There are several committees devoted to bridge elements including the Bridge Producers Committee and the Bridge Committee. PCI also has a Tolerance Committee. It is important to note that this committee coordinates

with many other committees including building committees. PCI publications are given significant scrutiny by all cross-cutting committees. Michael Culmo met with several PCI committees at the 2014 and 2016 PCI National Conventions. The following are the notes from these meetings:

1. Tolerance Committee:
    a. The Tolerance Committee does not set tolerances for PCI. They act as a clearing house for the other PCI committees that recommend tolerances.
    b. The Tolerance Committee manages the updates to the PCI Manual MNL-135-00, *Tolerance Manual for Precast and Prestressed Concrete Construction*. These updates are based on input from other PCI committees. This manual is largely intended for vertical construction; however, many of the tolerances can be applied to PBES products.
    c. The Tolerance Committee stated that, to the best of their knowledge, the *PCI Tolerance Manual for Precast and Prestressed Concrete Constructio*n values are not based on statistical data. To quote the Foreword of the manual: "This document is the compilation of over 50 years of Precast/Prestressed Concrete Industry Experience…" Note that they were not aware of a statistical analysis of previous experience; however, this does not necessarily mean that it was not done.
2. Bridge Producers Committee:
    a. This committee is the primary decision-maker for development of tolerances. This group reports to the Tolerance Committee on any proposed changes to the PCI Tolerance Manual.
    b. Michael Culmo discussed the need for bridge tolerances for PBES projects and the effect of tolerances on joint width.
    c. The committee noted that, in theory, they can produce products to any tolerance. Elements with very tight tolerances will be much more expensive due to several factors:
        i. Need for more expensive and durable formwork.
        ii. Need for additional QC checks during production.
        iii. Potential rejection of "out of tolerance" elements.
    d. The producers also stated that they fabricate to the specified tolerance (whatever it is). For instance, if a tolerance on length is $\pm$1/2", they will make that their goal. If the tolerance is less, they will be more careful to meet that tolerance. The reason that this is important is that comparing actual measurement data from one project to the next is invalid, since a different level of fabrication QC would have been applied. If statistical data is used, it needs to be from projects with equal tolerance specifications.
3. Bridge Committee:
    a. Michael Culmo presented information on this NCHRP Project. There was interest in the committee; however, the Bridge Committee is mostly responsible for design related issues, not fabrication.

The experts on these committees also noted tighter tolerances lead to stricter quality control in the shop. However, there is a limit to how much control can be exercised. If very tight tolerances are specified, the likelihood of elements being rejected increases. This aspect increases the risk to the fabricator which results in inflated costs that the fabricators pass on to the contractor and, ultimately, to the agency. The goal is to establish "reasonable" tolerances that can be produced without significant risk of rejection.

PCI also publishes a document entitled *Design and Typical Details of Connections for Precast and Prestressed Concrete* (PCI (1988)). This document contains information on tolerances for connecting precast elements together. As with the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction*, the basis of the recommended tolerances is industry experience.

The ACI also publishes a manual on tolerances entitled *Specification for Tolerances for Precast Concrete* (Tipping, Et al (2010)). This manual is very similar to the PCI Tolerance Manual, and is used primarily in the vertical construction industry. Both the PCI and ACI documents contain significant information on element and erection tolerances; however, the basis of these tolerances does not appear to be founded on actual production dimensions.

The FHWA manual entitled *Accelerated Bridge Construction – Experience in Design, Fabrication, and Erection of Prefabricated Bridge Elements and Systems* (FHWA (2011)) includes a section on tolerances in ABC construction. Several different types of tolerances were identified:

- Element Fabrication Tolerances: Variation in element length, thickness, flatness, etc.
- Dimensional Growth: The build up of total structure length brought on by tolerances when butting multiple elements side by side.
- Hardware Tolerances: Tolerances of embedments and protrusions from prefabricated elements.

The Northeast Region of the Precast/Prestressed Concrete Institute (PCINE) manages a bridge technical committee that has been developing ABC guidelines for many years. One recently revised guideline includes information on tolerances. This committee has studied other potential tolerance issues including:

- Erection tolerances: Ability to set a prefabricated element in the specified location
- Joint Width Tolerances: Specification tolerances for joint widths based on the adjacent element tolerances and the erection tolerances. In general, the committee recognizes that larger element tolerances equate to larger joint widths; and the converse, that tight joints require tight tolerances.

The PCINE Bridge Technical Committee has published a document entitled *Guidelines for Accelerated Bridge Construction Using Precast/Prestressed Concrete Elements Including Guideline Details*. This guideline includes recommendations for tolerance for common ABC elements and the accommodation of tolerances through the use of grouted joints. The tolerance portion of the guideline is based on the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (MNL-135-00)*.

Several research papers were located that discussed tolerances for deck panels. A paper entitled *Design Considerations of Full-Depth Precast/Prestressed Concrete Bridge Deck Panels* (Issa Et al) discussed tolerances, but did not give specific guidance on recommended tolerances. Another paper entitled *Transverse Joint Configuration Development and Testing for a Modular Bridge Deck Replacement* (Goodspeed (2011)) included laboratory testing of different joint configurations. The goal was to develop a joint that did not require grouting or splicing of tendon ducts. The approach was similar to match casting; however, the surfaces were formed. The results were that it was very difficult to get a solid match between panels without creating stress risers at high points. This work reinforced the conclusions of the PCINE Bridge Technical Committee that tight joints require tight fabrication tolerances.

In an effort to see what other industries are specifying with regard to tolerance, the team investigated a publication by the American Society of Mechanical Engineers. They have published a document entitled *Dimensioning and Tolerancing – Engineering Drawing and Related Documentation Practices* (ASME (2009)). A review of this document indicates that it is mostly used for drawing production and not for

tolerance engineering. Tolerances in mechanical products are more a function of movable parts and seals. This is quite different than the issues with large prefabricated products installed with cranes. Based on this, it was determined that document would not be applicable to this project.

There is a small amount of specific information on tolerances for bridge systems. There are several published documents that include tolerances for construction and setting of bridges.

1. The Utah SPMT Process Manual (2009) and the Structures Design and Detailing Manual (2015) have tolerance specifications for setting bridge superstructures. They include both horizontal and vertical tolerances for setting the bridge; however there is not significant detail on the construction tolerances of the actual superstructure. It is assumed that the design will include significant closure pours at the deck ends to accommodate these tolerances, or that the approach slabs will be moved with the bridge. Twist tolerances are also discussed in this manual. The intent is to have the design engineer, or the contractor calculate the allowable twist that the structure can accommodate without damage, which establishes the allowable twist tolerance. The contractor then develops a twist monitoring system to ensure that this twist factor is not exceeded.
2. The FHWA SPMT Manual (Ralls, 2007)) also contains information on construction and erection tolerances. In general, this manual calls for the contractor to establish tolerance criteria. This manual also recommends the use of larger closure pours to accommodate tolerances.
3. The SHRP2 Addendum to the ABC Toolkit (2013) covers lateral sliding techniques. This document does not contain significant guidance on construction and erection tolerances.
4. The FHWA slide-in bridge construction (SIBC) Implementation Guide (27) is similar to aforementioned Bridge System Manuals. There is discussion on different tolerances and management of them; however, there are no specific tolerance recommendations, aside from bearing elevation tolerances.

Tolerances are not only limited to precast concrete elements. There are tolerance specifications for prefabricated steel. Steel elements are always prefabricated; therefore the use of existing specifications is reasonable and recommended. The National Steel Bridge Alliance (NSBA) was contacted for information on tolerances. They indicated that the fabrication tolerances of steel bridge elements are well established. To date, most steel elements used in ABC projects have been beams and girders, therefore traditional tolerance specifications are applicable. The primary source of tolerance specifications is the AASHTO/AWS D1.5 Bridge Welding Code (AASHTO/AWS (2010)). The National Steel Bridge Alliance also published guidelines on steel bridge construction (AASHTO/NSBA (2002, 2008, 2014)). These guidelines offer additional guidance on fabrication and erection of steel bridge framing. If other steel elements were to be used (pier columns, caps, etc.), the fabrication tolerances used for beams would most likely be sufficient.

Conclusions

In general, the need for specifying tolerances is understood and agreed upon by the engineering community. There is only minor guidance on the actual tolerances that should be used. The detailing of the bridge will have an impact on specified tolerances. For example, if a cast-in-place concrete closure joint is detailed at the ends of the bridge deck, the need for very tight tolerances may not be justified. Likewise, if a bridge expansion joint is to be included in the bridge deck prior to the move, a very tight tolerance may be required. There is a need to strike a balance between reasonable tolerances and risk of elements not fitting together.

3.1.2 Dynamics of SPMT Bridge Systems

Very little research has been undertaken in the field of dynamics of bridge system moves. A few projects related to friction in bridge sliding systems were found, which was helpful in establishing criteria for this research. One such project was completed by the University of Washington (Taylor, J.C. and Stanton, J.F. (2010)) and another informal study was completed by Iowa State University.

The team has located several important documents on bridge system moves using SPMTs and SIBC. For the most part, these documents cover past projects and practices. CME Associates has also worked with several international heavy lift contractors. Through this experience, we have learned that there are certain dynamic features of bridge moves that are not well known, which primarily include:

- Vertical Accelerations during SPMT moves (bouncing)
- Horizontal Accelerations during SPMT moves (starting and stopping)
- Static and dynamic friction of sliding systems

Several literature documents offer recommendations on some of these values; however, there does not appear to be justification for the recommendations made except for contractor experience. The goal of this research project was to establish reasonable dynamic factors for bridge system moves based on actual testing.

Other countries have used SPMTs for moving bridges for construction projects. The U.S. Federal Highway Administration, AASHTO, and National Cooperative Highway Research Program (NCHRP2) sponsored a scanning team to conduct a study in Japan and Europe (Tang and Ralls (2005). This was implemented to help further knowledge in the United States of new and innovative ways for bridge construction. The team found that there are multiple heavy lifting companies in Europe which used SPMTs to move not only sections of bridges but even whole bridge structures at a time. Some of the bridges moved in Europe were large, requiring multiple SPMTs to move them, such as a 3600 ton bridge near Amsterdam's Schiphol Airport. This bridge move required SPMT with 134 axle lines.

The literature search did not reveal any international codes that have specifications for SPMT bridge moves. Several international heavy lift companies and experts were also contacted. They were not aware of any international specifications for SPMT moves. Each company designs their systems using experience from past moves.

The Utah DOT *SPMT Process Manual and Design Guide* (2009) does contain requirements for vertical dynamic load effects, by recommending amplification of forces in the superstructure by 15% to account for vertical dynamic effects. This recommendation is based on data collected by UDOT contractors during 5 bridge moves in 2008. Researchers from Utah State University (USU) used the data for further study to help determine the dynamic effects on the bridges (Rosvall, Et al (2010)). Upon analyzing the data obtained, they made several conclusions that were incorporated into the Utah DOT Guide. This guide does not contain recommendations for horizontal dynamic effects because the data gathered by the contractors did not capture horizontal effects. The measured dynamic stresses were relatively small in comparison to the yield strength of the structural steel, but when they were compared to the lifting stresses and the change in stress, the dynamic stresses were significant. The ratio of dynamic stresses to lifting stresses depends greatly on the length of the cantilever beyond the lifting points. Because of this, it was recommended that additional research be done so that stronger conclusions can be made.

FHWA publication entitled *Manual on Use of Self-Propelled Modular Transporters* (2007) offers significant guidance on the use of SPMTs; however, there are no recommendations for accounting for vertical or horizontal dynamic effects.

There is currently little knowledge base for the effects imparted on bridge structures when being moved using SPMTs. Some designers have considered a SPMT bridge move to be a quasi-static load event. The Utah studies noted above contradict this assumption. The vertical accelerations caused by driving over uneven ground could have an effect on the structural integrity of the bridge if the dynamic effects are significant. For example, vertical dynamic effects can cause deck cracking if the dead load deck stresses are close to the modulus of rupture of the deck concrete. Horizontal dynamic effects would theoretically not have an effect on the superstructure; however, the effects on the supporting falsework can be significant.

The team has worked with and contacted several heavy lift companies including Mammoet, Sarens, and Barnhart. These companies make up a significant portion of the suppliers of SPMTs in North America. CME has also completed several heavy lift projects with these companies. In general, the companies design their systems with rule-of-thumb engineering. This applies to both vertical and horizontal forces. To date, none of the companies have completed rigorous testing of actual SPMTs for vertical and horizontal dynamic forces. They have, on occasion, attached accelerometers to power plant loads at the request of the owners of the equipment. This was done primarily for insurance purposes and the data was not shared with the heavy lift companies.

Conclusions

There is no consensus document regarding dynamic effects of SPMT bridge systems. Methods for calculating dynamic effects are primarily based on rules of thumb and the experience of heavy lift companies. With more companies entering into the ABC market, the level of experience may be reduced. Therefore, there is a growing need to standardize methods for SPMT dynamic effects.

3.1.3 Dynamics of Lateral Slide Bridge Systems

Lateral slides, also known as horizontal slides, SIBC, or horizontal skidding, consist of building the bridge superstructure on temporary piers next to the current bridge it will replace, and then sliding it into place once the old one has been removed. According to the Wisconsin DOT Bridge Manual (2013) "One common method is to push the bridge using hydraulic rams as the bridge slides on a smooth surface and Teflon® coated elastomeric bearing pads. Other methods have also been used, such as using rollers instead of sliding pads, and winches in place of a hydraulic ram" (Kim, Et al (2011)). The Strategic Highway Research Program 2 (SHRP2) has created an addendum to the *Innovative Bridge Designs for Rapid Renewal: ABC Toolkit on Lateral Slides* (SHRP2 (20130)), which describes the applicability of using the method, design considerations, and key components when implementing a bridge slide. It also describes the different types of sliding devices that can be used: rollers, slide shoes, and slide plates. Typically Teflon® (PTFE) and stainless steel are used on the contact surfaces to reduce friction. Sometimes lubricants like dish soap are used to reduce the friction. The SHRP2 report contains recommendations for static and dynamic friction; however, it is noted as being based on project experience, not research.

FHWA had sponsored the development of an implementation guide for SIBC. It was developed by the UDOT and the Michael Baker Corporation (FHWA (2013)). The SIBC guide was designed to demonstrate the advantages of using SIBC and show how it can be implemented by state and local agencies. This guide also contains recommendations for static and dynamic friction; however, it is believed that these values are also taken from project experience and not research.

Iowa DOT put together an ABC workshop which was held August 11-12, 2008 in Des Moines, Iowa (Ralls (2008). The workshop brought together officials from Iowa DOT, many other state DOTs, Federal Highway Administration, NSBA, Iowa State University, local engineering consultants and contractors, and many other professionals involved in ABC projects or research. The workshop was invitation only and had over 90 participants. Multiple presentations were given in order to bring all the participants up to date on ABC projects across the U.S. and current ABC methods. After several hours of presentations in the morning, groups were created to facilitate collaboration throughout the afternoon. During these workshops, groups brainstormed ideas of how ABC methods could be used, how to improve prefabricated design details, and then discussed opportunities and obstacles for their ideas. The groups also discussed research that needed to be done, policy changes, and areas where specifications could be developed. Ideas and methods for several of Iowa's upcoming bridge projects and what action could be taken to implement ABC on those projects were shared (22). The workshop report contains information on various lubricants and sliding friction; however, it is based on project experience and not research.

In December of 2013, Michigan DOT (MDOT) also held an ABC workshop (Aktan et al. (2014). The workshop was organized for reasons similar to the workshop held in Iowa. In 2008, prefabricated bridge elements were used for the first time in Michigan. Following the success of the project, MDOT built several other bridges using the same method. MDOT wanted to expand the use of ABC techniques to incorporate bridge slides and the use of SPMTs. The purpose of the workshop was to leverage knowledge and experience to see how they could use these innovative techniques for the design and construction of future Michigan bridges. Some of the information shared about bridge slides included static and dynamic coefficients of friction and how MDOT would use the coefficients to select required jacking hydraulics to move a bridge structure. Often with slide-in systems, Teflon®-laminated neoprene steel reinforced pads are used on a track system. It is set up so that the Teflon® surface is in contact with stainless steel coated carriages to produce low friction coefficients. The group discussed lateral restraints that are needed to keep the bridge structure aligned, as well as the complications of sliding a multi-span structure.

The Wisconsin Department of Transportation sponsored research by the University of Washington (UW) to conduct tests for determining coefficients of friction between several different stainless steel finishes and a Teflon® (PTFE) coated surface (Taylor and Stanton (2010)). The purpose of the report was to provide the Wisconsin DOT with the results in a clear, straightforward manner and to be implemented by the Bureau of Structures for bridge design. Bearings used for bridge slides are often made with sheet Teflon® (PTFE) and stainless steel polished to a #8 mirror finish. There are many different types of surface finishes for stainless steel. A #8 mirror finish is much smoother and glossier than a 2B rolled finish. AASHTO LRFD Bridge Design Specifications use a #8 finish for permanent bearings. Since stainless steel polished to a #8 finish is more expensive and can be more difficult to obtain than other finishes, Wisconsin DOT had UW test 2B rolled finish and rough as-rolled finish stainless steel, and then compare the results to #8 polished stainless steel. The report explains the most important parameters that cause friction to vary when using PTFE and stainless steel. They are surface finish, contact pressure, sliding speed, length of slide path, and temperature. The tests conducted in the study addressed all of those factors except for temperature because special equipment is required for low temperature testing.

Most of the results obtained in the study produced expected results, but the slide path test produced consistently counterintuitive results. The coefficient of friction for the #8 mirror finish increased as the slide path increased, whereas for the 2B rolled finish it stayed constant, and for the rough as-rolled finish it decreased. From these tests, an equation was developed to help predict the friction coefficient as a function of the surface finish, the contact pressure, the sliding speed and the length of slide path. After all of the tests were completed and the results analyzed, they determined that stainless steel with a 2B surface finish is an appropriate alternative to a #8 mirror finish. However, it is important to note the tests did not

analyze how temperature change affects the coefficient of friction, which is one of the important parameters that should be considered.

Another ABC method of constructing bridges is the Incremental Launching Method (ILM). This method is one that has been around for many years. Many European bridges have been constructed using this method. In the past, ILM has been used little in the United States. However, its use has been increasing recently in the U.S. as it proves to be a very effective way to build a bridge in areas with site restrictions such as deep valleys, water crossings or environmentally-protected areas.

In 2007, a report was created entitled *Bridge Construction Practices Using Incremental Launching* (LaViolette, Et al (2007)). The purpose was to report available information about ILM, applications of this method, limitations, and benefits. The group conducted a comprehensive literature review for ILM which is contained in the report. They also provided recommendations for bridge owners, designers and contractors regarding the best practices for planning, designing and construction techniques using ILM, and explain possible applications and limitations. While replacing a bridge over the Iowa River, Iowa DOT implemented an incremental bridge launch for a girder bridge and monitored multiple parts of the substructure and superstructure during the launch (Wipf, Et al (2004)). Monitoring data included strain, deflection, tilt and loads. The bridge was erected in 6 full-span launches and data was collected for all of the launches. The data was used to monitor the flexural behavior of the girders, contact stress between the girders and bridge roller bearings used to launch the bridge, global and local pier column behavior, and cross-frame force distribution and magnitudes.

When looking at the pier columns, residual or "locked in" stresses built up at the end of the day. There were extremely large contact stresses measured on the bottom flange and web of some of the girders as they rolled over the temporary supports. Based on the results obtained and the experience of launching the bridge, there were a few recommendations made. They advised that the design of the girders should include the girder-to-roller contact stresses, which can be very high when launching a bridge. To reduce these high localized stresses in the girders, they suggest using either larger diameter rollers or multiple small rollers, which can spread the load. Other advice was with regard to the design. They recommended considering the potential to have unequal vertical bearing forces applied from the girders to the supports as the structure is being launched. This is due to the potential misalignment of the girders causing them to not be perfectly centered between the supports. It is important to recognize that these uneven forces may cause warping of the bridge structure as a whole during the launch and that measures should be taken to prevent this from happening.

The FHWA manual entitled *Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems* (Culmo (2011)) included the "State of the Practice" for all aspects of ABC as of 2011. It covers what Accelerated Bridge Construction is and what the benefits are when using ABC technologies. It fills in many of the gaps where information is lacking in previous manuals. The Structural Placement section describes 5 ways in which the structure can be moved into place. These 5 methods include: SPMTs, longitudinal Launching, Horizontal Skidding or Sliding, Other Heavy Lifting Equipment and Methods, and Conventional Cranes. For each method, an overview is given of the process. Examples are used to illustrate the usefulness of the method, and narratives describe the types of situations in which each method excels. Each of these methods has parameters and limitations that must be considered when selecting the bridge placement method to be used, and this manual helps decision-makers with that choice.

One international research paper was discovered during the exploration. This paper entitled *Coefficient of Friction in the TFE Slide Surface of a Bridge Bearing* (Campbell and Manning (1990)) covers

laboratory testing that was done in Ontario, Canada in 1990 by Queens University and the Ontario Ministry of Transportation. The testing was undertaken to verify friction coefficients for permanent bridge bearings. They investigated several parameters in TFE/stainless steel bearing surfaces including: contact pressure, temperature, sliding speed, roughness, and contamination of the sliding surface.

Conclusions

As with SPMTs, there is no consensus document regarding dynamic effects of Lateral Slide Bridge Systems. There has been limited research regarding sliding systems; however the work was limited to a few sliding systems. There is a need for specifications for commonly used sliding systems

3.2 Insufficiencies in Existing Guidelines and Manuals

3.2.1 Tolerances

There is limited information on tolerances in the various published manuals and guidelines covering ABC and PBES. The most significant document that was found is the *PCI Tolerance Manual (MNL-135-00)* (PCI (2000)). This manual has been used by the PCI Northeast Bridge Technical Committee to develop typical recommended tolerance details for commonly used precast elements (Seraderian, Et al). The basis of this work was the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction* with adjustments that were decided upon in a committee that included bridge fabricators.

The *PCI Tolerance Manual* does not give much guidance on the fit-up of precast products and the role of joints. PCI has also published a manual entitled *Design and Typical Details of Connections for Precast and Prestressed Concrete* (PCI (1988)). This manual offers some guidance on tolerances related to connections. This manual is based on contractor experience in the vertical construction industry; however it does provide more information as a starting point for the development of a tolerance guideline on connections. The PCI Northeast Bridge Technical Committee has developed methods for specifying joint width tolerances that account for the tolerances of adjacent elements and their erection tolerances. Equations were developed based on the potential for additive build up of tolerances by assuming that each element is built to the maximum tolerance. This does not account for the probability that both elements are at the maximum tolerance. It may be feasible to reduce the specified joint widths using the probability that both elements are not fabricated to the maximum specified tolerance. The team will investigate these probabilities in subsequent phases of this project and make recommendations to the project panel.

The following is a listing of the insufficiencies that were found:

- All published specifications for element tolerances are based on experience, not actual data.
- The approach for specifying tolerances for connections is limited.
- There recommended tolerances for bridge moves are limited.
- There may only be limited amount of fabrication and installation data available to study.
- There is a need to verify past tolerance recommendations through the use of statistical data. Once a statistical approach is developed, it can be used to develop future tolerance recommendations when data is available to analyze.

The approach for addressing these insufficiencies under this research project is to establish reasonable tolerance recommendations for prefabricated elements based on statistical data. Chapter 4 of this report describes the proposed methodologies and approach in more detail.

Based on the literature review, the general conclusion is that existing tolerance specifications are based on the years of experience of fabricators and contractors. This is not necessarily a bad approach. Setting tolerances based on experience is a similar to performing a statistical analysis. Experience is an indirect way of achieving the same result. Through experience, fabricators and contractor will learn what can be fabricated and built within reasonable certainty on a regular basis. Statistical analysis of data will verify the probability of success to a measured degree.

3.2.2 Dynamics of SPMT Bridge Systems

There are several very useful documents that have been published for SPMT bridge systems. These include the FHWA document entitled *Manual on Use of Self-Propelled Modular Transporters to Move Bridges* (Ralls, (2007)) and the Utah DOT document entitled *Accelerated Bridge Construction SPMT Process Manual and Design Guide* (UDOT (2009)). These documents offer some guidance on dynamics that are based primarily on past experience and recommendations from heavy lift contractors.

The Utah manual does offer some guidance on vertical dynamics that are based on actual bridge move data. During the development of the guide, the Utah DOT engaged USU to examine data that have been collected from a number of bridge moves using SPMTs. The data was collected by the contractors prior to the involvement of the Utah State researchers; therefore, the data was not perfect. The locations of gages could have been better situated and there was no measurement of horizontal dynamics. The Utah DOT had the USU investigate the data (Rosvall, Et al (2010)). The results of this investigation led to the dynamic guidance that is in the final Utah SPMT Process Manual. This guidance is based on limited data; therefore, proposed work under this project is justified.

3.2.3 Dynamics of Lateral Slide Bridge Systems

As with publications regarding SPMT bridge systems, there is little guidance regarding dynamics of Lateral Slide Bridge Systems. The nature of the equipment used to move bridges in this manner is quasi-static. The rate of movement is measured in inches per minute, versus feet per second. Another main difference between SPMT and Lateral Slide Bridge Systems is that the structure is typically supported very close to the permanent girder supports, making vertical dynamic impacts insignificant. Based on this point, engineering judgement would indicate that vertical and horizontal dynamic factors are not of consequence in Lateral Slide Bridge Systems.

The one factor that is of consequence is the static and dynamic friction of the sliding systems. Several publications have been issued regarding Lateral Slide Bridge Systems including the FHWA document entitled *Slide-In Bridge Construction Implementation Guide* (FHWA (2013)) and the SHRP2 publication entitled *ABC Standard Concepts: The Lateral Slide –Addendum to Innovative Bridge Designs for Rapid Renewal* (SHRP2 (2013)). Both of these documents contain recommendations for friction values; however, these are based on previous experience and manufacturer's recommendations, not actual laboratory data (these documents did not include the results of the research noted in Section 3.1.3 of this report). It is important to know the friction values for these systems in order to properly design the slide system. Lack of knowledge of these details can lead to situations where slide systems fail, resulting in unwanted project delays. It is also important for the designer to know how much force is being imparted on the bridge so that it can be checked to ensure that there will be no damage sustained during the move.

The proposed research for this this project focused on supplying laboratory testing of various potential slide system materials. The team focused on cost-effective materials that provide acceptable performance.

CHAPTER 4

Development of Tolerances for Prefabricated Elements and Systems

4.1 Introduction

One of the goals of this research is to verify the ability of producers to make products that can be fabricated within the specified tolerances, and to develop tolerances that are reasonable and repeatable without incurring large fabrication costs. The tolerance values that are published are based on past fabrication history. Statistical analysis of actual fabrication data can be used to establish tolerances that are based on a preset acceptability rate (such as 95% confidence). This research will establish a statistical method that can be used industry-wide to verify and/or establish future tolerance specifications.

The list of elements that will be included in the guideline include:

- Full-Depth Deck Panels
- Pier Caps
- Pier Columns
- Wall Elements (abutments, piers, integral abutment stems)
- Footings
- Modular Deck Beam Elements

These represent the most common prefabricated elements in use today. It is anticipated that additional elements will be developed in the future. Agencies and engineers should be able to base tolerance specifications from elements in this list that are of similar size and shape. For example, a precast abutment seat cap could be fabricated to the same tolerances as a pier cap.

4.2 Methodology for Statistical Analysis of Fabrication Data

Standard statistical methods can be used for the analysis of data from fabrication plants. In probability theory, normal (or Gaussian) distribution is defined as a set of data that conforms to commonly occurring distribution. Observed deviation from specified dimensions should, in theory, follow this type of distribution since the fabricators are striving for a specified dimension. However, the result can vary based on commonly occurring inherent variations in the fabrication process (form fabrication, form wear, shrinkage of concrete, etc.). For a normal distribution, the probability that any data point falling between any 2 limits can be calculated for that distribution.

If a normal distribution does not model the observed deviation from specified dimensions well, alternatives exist. The critical tails of the distribution of deviation, where tolerances will be set, can be modelled as normal, ignoring the rest of the distribution. This alternative was employed recently to calibrate the fatigue limit states of the *AASHTO LRFD Bridge Design Specifications*.

There are several key terms used in normal distribution data analysis:

1. Standard Deviation: The amount of dispersion of the sample values from the average value. It is calculated by taking the square root of the average of the squared differences of the values from their average value.
2. Mean or Average: The most likely value of the distribution. It is calculated as the sum of the values in the sample divided by the number of values.
3. Coefficient of Variability (COV): The amount of variation of the sample, calculated as the standard deviation divided by the mean (a dimensionless quantity).
4. Bias: The mean of the sample divided by the target value.

The uncertainty of a distribution is usually represented by the bias and the COV. A worn form would yield a biased distribution where the observed deviation would be centered not on the target dimension, but centered on the over-dimension of the worn form.

A target reliability index, β_T of 2.0, or a 95% probability of success, should be acceptable for plant production of prefabricated products. This is assuming that a 5% failure rate of element tolerance is a reasonable value to strive for on most bridge products. Based on this, the analysis of fabrication data would use the following procedure:

1. Obtain data values that represent the deviation from specified dimensions for a population of products made using the same tolerance specification.
2. Sort the occurrence of the data into specified bins, and plot that data set using a histogram.
3. If the data conforms to a normal distribution, calculate the mean and standard deviation of the data set.
4. If the data does not conform to a normal distribution, create a fictitious normal distribution using only the critical tail of the actual distribution. Calculate the mean and standard deviation of the fictitious distribution.
5. Calculate the 95% confidence limit for the data set by adding and subtracting 2 standard deviations from the mean.
6. Observed biases may need to be considered in the final selection of tolerances.

Figure 4.2-1 contains an example of this process:

Example of Determining Apprpriate Tolerance Based on Statistical Analysis

Tolerance Data

0.22
0.24
0.12
0.1
-0.2
-0.22
-0.05
-0.06
0.04
0.02
0.21
-0.23
0.02
0
-0.01
-0.21
-0.14
-0.08
-0.06
0.03
0.1
0.01
0.03
-0.07
-0.03
0
-0.1
0.2
-0.06
0.01
0.28
-0.1
0.15
-0.28
0.03
0
-0.02
0.05
-0.15
0.12
0.01
-0.02
0
0.03
-0.01
0.02
0
0.15
-0.23
0.03

Standard Deviation	0.125962
Mean	-0.0022
Mean plus 2*SD =	0.249725
Mean minus 2*SD =	-0.25412
Tolerance Result =	± 1/4"

95% of the data fits within this value

Bin	*Frequency*
-0.4	0
-0.3	0
-0.2	6
-0.1	4
0	16
0.1	15
0.2	5
0.3	4
0.4	0
More	0

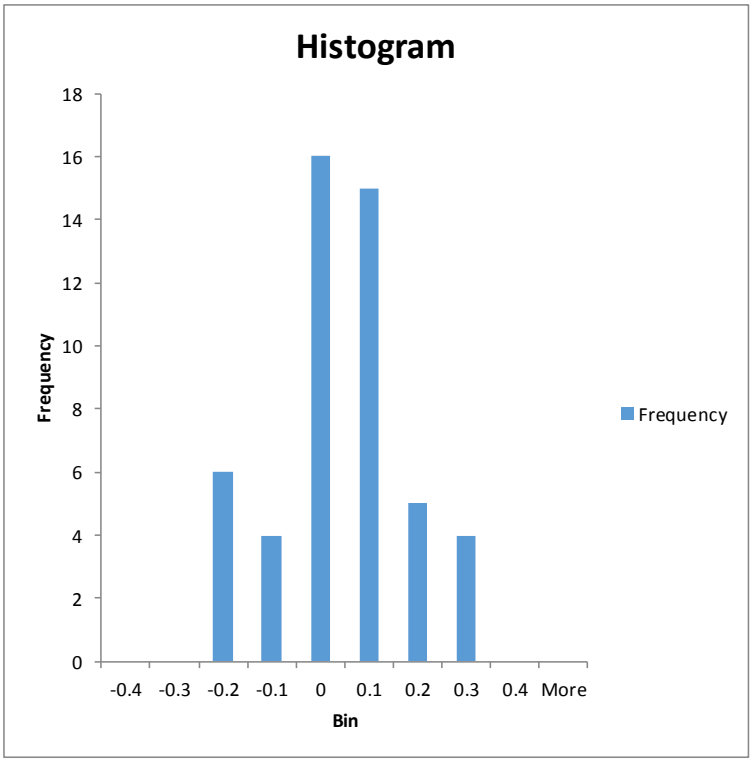


Figure 4.2-1 Example of Statistical Analysis of Tolerance Data Set without Bias

The 95% one-sided confidence limit is demonstrated by having only 2 out of 50 data points outside the 2 standard deviations. In this example, the mean was very close to zero; in other words, the bias is very close to zero. It is possible that the mean may be slightly higher or lower than the target value, but it is typically higher due to phenomenon such as worn forms. The observed biases may suggest that the selected tolerances should consider some limited degree of bias.

Figure 4.2-2 shows an example data set where this could be the case.

Example of Determining Apprpriate Tolerance Based on Statistical Analysis

Tolerance Data

0.44
0.24
0.15
0.15
-0.2
0.41
0.04
-0.06
0.24
0.02
0.26
-0.23
0.15
-0.11
0.32
-0.21
0.13
0.13
-0.06
0.24
0.1
0.08
0.21
-0.02
0.25
0.15
-0.1
0.24
-0.06
0.1
0.28
-0.1
0.21
-0.05
0.1
0.24
-0.02
0.41
-0.07
0.28
0.21
-0.02
0.15
0.28
-0.01
0.02
0
0.15
-0.1
0.03

Standard Deviation	0.164559
Mean	0.0998
Mean plus 2*SD =	0.428917
Mean minus 2*SD =	-0.22932
Tolerance Result =	± 0.43", -0.23"

Bin	*Frequency*
-0.5	0
-0.4	0
-0.3	0
-0.2	3
-0.1	4
0	10
0.1	8
0.2	8
0.3	13
0.4	1
0.5	3
More	0

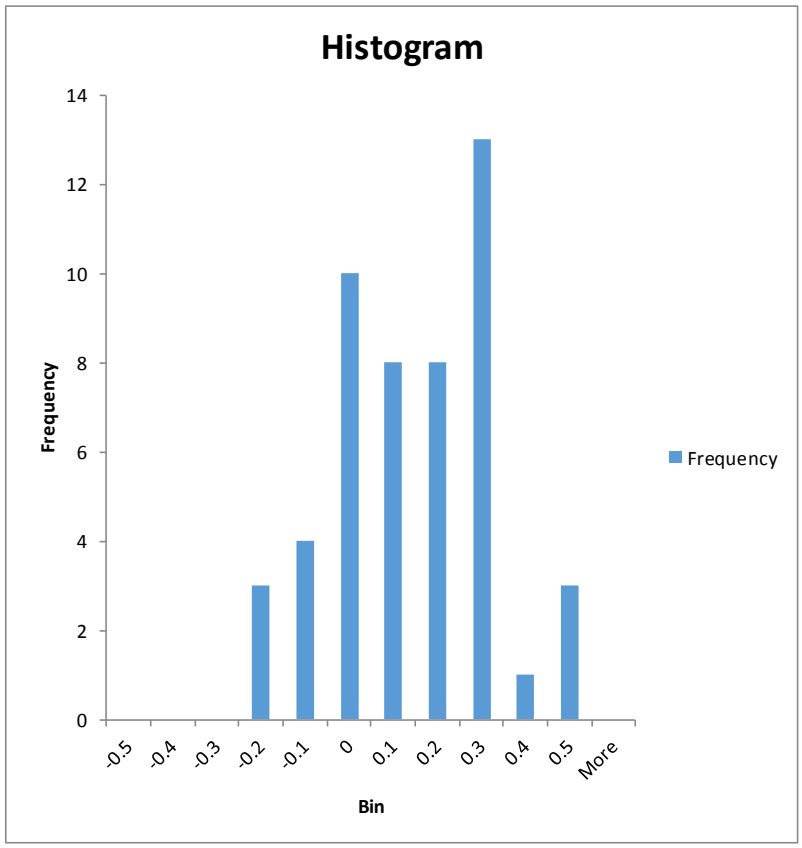


Figure 4.2-2 Example of Statistical Analysis of Tolerance Data Set with Bias

In this example, the mean is approximately 0.1". The mean plus 2 standard deviations is 0.43", while the mean minus 2 standard deviations is -0.23. This shows that the product is consistently larger than the specified or biased value. It demonstrates a pattern of a consistent positive variation that could be attributed to a factor such as stretching of the forms due to frequent use. The specification tolerance based on this data could account for this, since it may be a common occurrence.

4.3 Collection of Tolerance Data

The initial goal of this project was to analyze actual data and compare the results with the existing published tolerance manuals, specifically, the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000)*. This manual was developed by a committee at PCI based on years of experience fabricating products. The tolerance values in the manual are set to values that are commonly attainable in normal production. In reality, the producers honed in on tolerances without a mathematical back-up. The methodology noted in Section 4.2 was developed to put math to the process of determining realistically attainable tolerances. The process was based on a 95% acceptance rate. The process could easily be modified to accept more or less acceptance by adjusting the number of standard deviations from the mean.

The collection of data for analysis proved to be problematic. The team has been in contact with a number of precast fabricators regarding the availability of data. The team also contacted members of the PCI National Bridge Producers Committee requesting data to analyze. The results of this search are as follows:

a. Responses were limited
b. Several fabricators checked their files and found that their quality control staff only checks tolerances using a pass fail process. If the piece is within tolerance, it is noted as such. No measurements are recorded.
c. One producer submitted very basic data from one project. This data was analyzed using the proposed methodology and is presented below.
d. Several producers questioned the need for an ABC PBES tolerance manual, since PCI already has a tolerance manual *(PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000))*, that has been in use for many years. The thought was that a piece of precast is a piece of precast regardless of where it will eventually be installed (building, stadium, bridge, etc.). They felt comfortable using the existing PCI Tolerance Manual would be sufficient for tolerance specifications, since it has been in use for many years.

There is only one set of data was submitted to the team by Central Premix in Spokane, Washington. The data was for vertical wall panels for a building project. It is not an ideal data set since it is only contained 2 measurements and it is for a building project, not a bridge project. The following is the data that was submitted:

Data taken from 159 casted pieces on Pasco Police.

Dimensions that only exceeded 1/8" were documented.

(9) pieces: length +1/4"
(20) pieces: length -1/4"
(5) pieces: length -3/8"
(9) pieces: width +1/4"
(3) pieces: width -3/8"

The team had to make certain assumptions for the panels that were not reported. The variation used for the analysis was set at 0" based on the fact that all of these panels were close to zero in variation. Based

on this assumption, a statistical analysis was performed based on the methodology described in Section 4.2. The results are shown in in Figures 4.3-1 and 4.3-2.

Fabricator: **Oldcastle Precast, Spokane, WA**
Element: **4" Thick Panels, 2'-4" to 5'-8" wide, 3'-11" to 17'-6" long (Architectural Panels)**
Measurement Type: **Panel Length**
Number of Data Points: **159**

Data Information

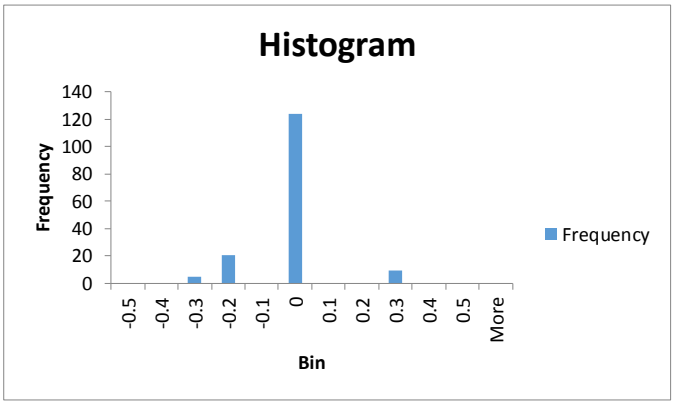
Data taken from 159 casted pieces on Pasco Police.

Dimensions that only exceeded 1/8" were documented.
(9) pieces: length +1/4"
(20) pieces: length -1/4"
(5) pieces: length -3/8"

Results:

Standard Deviation	0.123981
Mean	-0.03066
Mean plus 2*SD =	0.217301
Mean minus 2*SD =	-0.27862
Rounded Tolerance Result =	+1/4"
	-1/4"

Bin	*Frequency*
-0.5	0
-0.4	0
-0.3	5
-0.2	21
-0.1	0
0	124
0.1	0
0.2	0
0.3	9
0.4	0
0.5	0



Source: Oldcastle, Spokane, Pasco Place Project

Figure 4.3-1 Tolerance Analysis for Wall Panel Length

Fabricator: **Oldcastle Precast, Spokane, WA**
Element: **4" Thick Panels, 2'-4" to 5'-8" wide, 3'-11" to 17'-6" long (Architectural Panels)**
Measurement Type: **Panel Width**
Number of Data Points: **159**

Data Information

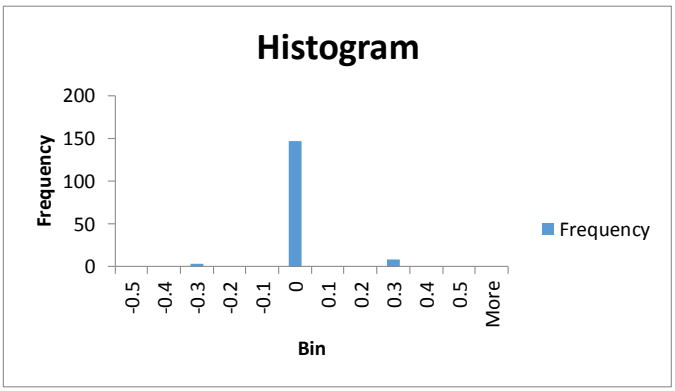
Data taken from 159 casted pieces on Pasco Police.

Dimensions that only exceeded 1/8" were documented.
(9) pieces: width +1/4"
(3) pieces: width -3/8"

Results:

Standard Deviation	0.078612
Mean	0.007075
Mean plus 2*SD =	0.164299
Mean minus 2*SD =	-0.15015
Rounded Tolerance Result =	+3/16"
	-1/8"

Bin	*Frequency*
-0.5	0
-0.4	0
-0.3	3
-0.2	0
-0.1	0
0	147
0.1	0
0.2	0
0.3	9
0.4	0
0.5	0



Source: Oldcastle, Spokane, Pasco Place Project

Figure 4.3-2: Tolerance Analysis for Wall Panel Width

The data analyzed is not ideal. The large amount of data that is reported as "less than 1/8 inch" is not desirable. It would be better to have actual measurements in order to get a more accurate tolerance value. If most of the data points were nearly -1/8", the mean and the standard deviation values would be slightly changed.

The data was compared to the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000))*. Table 4.3-3 shows a comparison of the results of the analysis and the recommended tolerances in the PCI Manual. The analysis demonstrates that the results are similar to what is typically specified in the precast concrete industry.

Tolerance Measurement	Analysis Result	PCI Tolerance Manual
Architectural Panel Width (0 to 10 feet wide)	+3/16”	+1/8”

Table 4.3-2 Comparison of Sample Tolerance Analysis with PCI Tolerance Manual

It should be noted that this is a very limited comparison of data; however the basis for this analysis is considered valid. A simple statistical analysis of data where the tolerances are set at 2 standard deviations from the mean will encompass 95% of the data, which is a recommended limit for tolerances.

4.4 Tolerance Questionnaire

In lieu of gathering data, the project team recommended that the proposed guideline be based on existing publications. The *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000))* was chosen because it provided tolerance specifications for elements that closely matched common bridge elements. For instance, tolerances for a building column can be used for tolerances for a bridge column, since the tolerances are based on precast fabrication practices. A typical fabrication facility would be making both elements using the same forms and processes; therefore the tolerances would be the same. The PCI Northeast Bridge Technical Committee successfully developed tolerance specifications using this approach.

The project team wanted to ensure that there was national industry consensus on this approach. A questionnaire was developed and was sent to each State Transportation agency, PCI Bridge producers, and industry experts. The goal of the questionnaire was to show the recommended tolerances for common bridge elements and ask for concurrence or recommended changes. The results of the questionnaire are included in Appendix A.

The conclusions of this questionnaire were that the recommended tolerances are acceptable. The majority of responders agreed with the recommended tolerances, some requested larger tolerances and others requested smaller tolerances. Without a strong consensus either way, the recommended tolerances should provide a guideline that is accepted and reflective of industry practice. The guideline document was written to address the persons that requested different tolerances. Owners and designers can specify tolerances that differ from the recommended tolerances. The section that covers joint width tolerances gives guidance on how to manage larger or smaller tolerances (see Section 4.6).

4.5 Element and Erection Tolerances

The tolerances included in the final guideline document are based on the *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000))*. The tolerance questionnaire results indicated that there is industry agreement with this approach. The PCI Manual does not have a significant number of tolerances for bridge elements; therefore research team derived tolerances for bridge elements that most closely matched elements for buildings. For example, tolerances for bridge columns were based on tolerances for building columns. Likewise, tolerances for erection of prefabricated elements were based on similar building elements included in the PCI Manual.

4.6 Joint Tolerances

The *PCI Tolerance Manual for Precast and Prestressed Concrete Construction (2000))* is a very good document that is useful in developing tolerance guidelines for element fabrication; however, there is limited information on the width of joints. Joints between elements serve 2 functions:

1. They provide a space to install filler materials that are either used to seal the joint or make a structural connection
2. They provide room for adjustability of elements during erection to accommodate element tolerances

Both of these functions are important and need to be accounted for in specifying joint tolerances. There is a direct correlation between element tolerances and joint width. Larger tolerances require larger joint widths to accommodate them. Smaller joint widths require smaller tolerances. The goal of specifying joint widths is to have a joint that will have sufficient width to install filler materials and not have conflicts between connected elements, but not too large so that the appearance of the structure is compromised.

For any given joints, there are a number of element and erection tolerances that will affect the final joint width. For example, a vertical joint can be affected by the following tolerances:

- Width of adjacent elements
- Erection tolerances
- Edge flatness of the adjacent elements
- Squareness of the adjacent elements

Taking these variables, an equation can be written that defines the variable that affect the width of a joint. The following is an equation for a vertical joint between 2 wall panels:

$$T_{jw} = \pm\,[0.5(B_L + B_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_L + H_R)] \qquad \text{(Eq. 4.6-1)}$$

Where:

T_{jw} = vertical joint width tolerance
B_L = fabrication width tolerance of left element (in.)
B_R = fabrication width tolerance of right element (in.)
D_L = fabrication end skew/squareness tolerance of left element (in.)
D_R = fabrication end skew/squareness tolerance of right element (in.)
H_L = fabrication smoothness tolerance of left element (in.)
H_R = fabrication smoothness tolerance of right element (in.)
L_{HL} = horizontal erection tolerance of left element (in.)
L_{HR} = horizontal erection tolerance right element (in.)

The ½ term in this equation is based on the assumption that ½ of these tolerances are on each side of the adjacent elements.

One simple approach to specifying joint width tolerances is to assume that all of these tolerances are at the maximum at the same time. This is a very conservative approach; however the results would be very large joints, which is undesirable. The reality is that the probability of all tolerances being maximized at any given joint is quite remote. Therefore, there is a need to account for the probability of multiple tolerances occurring simultaneously. The statistical method used to account for this is Monte Carlo simulations. Monte Carlo simulations rely on repeated random sampling of data to obtain numerical

results of a mathematical combination of random variables. Monte Carlo methods can be used to solve any problem having terms that are variable and can be defined by a probability based distribution.

In a Monte Carlo simulation, random values for each term are generated based on the probability distribution of each variable. These values are entered into the equation and a result is generated. This process is repeated thousands of times in order to account for all potential outcomes. The results of these outcomes are a probability distribution for the results of the "equation". If the results follow a normal distribution, standard probability theory can be applied to the results. As with the process described in Section 4.2, a process can be developed to develop a joint width tolerance specification that is reasonable and attainable.

The variables in joint width equations are all based on tolerances that were previously specified. Since significant data was not available for analysis (see Section 4.3), the team was required to make certain assumptions for the Monte Carlo simulations. If one assumes that the specified tolerances represent a 95% confidence limit, the tolerances would represent 2 standard deviations from the mean. Using this, the probability distribution of each tolerance variable would be:

$$Mean = (Maximum\ tolerance + Minimum\ tolerance) / 2 \qquad \text{(Eq. 4.6-2)}$$

$$\text{Standard Deviation: } (|Maximum\ tolerance| + |Minimum\ tolerance|) / 4 \qquad \text{(Eq. 4.6-3)}$$

The value of 4 is based on the fact that 2 standard deviations from the mean will encompass 95% of the data in a normal probability distribution. This approach was used for each typical joint in a prefabricated bridge. An equation was developed using simple logic as to which tolerances would affect the width of a joint. The probability distribution of each tolerance was defined using Equations 4.6-2 and 4.6-3. A Monte Carlo Simulation computer program was then employed. The program will generate random values for each variable based on the defined distribution, calculate the result of the equation, and then repeat the process many times. The number of cycles of calculations will affect the outcome. If only a few cycles are used, the results will not be accurate.

The team ran multiple analyzes for each equation, varying the number of simulations, in order to determine the effects of the number of simulations on the results. The goal was to determine a reasonable number of simulations that achieves a desired accuracy. The number of simulations was varied from 125 to 5000. In all cases, the results generated a normal distribution of data. The mean and standard deviation resulting data sets were calculated. The percent change in the standard deviation between each analysis was also calculated. Figure 4.6-1 shows the results of one of the analyses. The standard deviation variation converges as the number of simulations increases. Convergence was defined when the percent change in standard deviation was less be less than 1% when compared to the previous analysis.

Number of Simulations	125	250	500	1000	2000	4000	5000
Mean	-0.01466	-0.00437	-0.00983	-0.00481	-0.00218	-0.00047	-0.00086
Standard Deviation	0.09066	0.08899	0.09147	0.09290	0.09071	0.08944	0.08921
Percent change in Standard Deviation		-1.84%	2.78%	1.56%	-2.35%	-1.41%	-0.26%

Figure 4.6-1 Simulation Variance Analysis

The results indicate that 5000 simulations will generate the desired variance of 1%. Running more simulations would essentially yield the same results; therefore additional simulations were not necessary.

This process was completed for each equation. The value of 5000 simulations was found to be adequate for each case.

Once a Monte Carlo simulation is completed, a tolerance for joint width can be specified. If a 95% confidence result is desired, the tolerance would be the mean, plus or minus 2 standard deviations. While Monte Carlo simulations are a very useful tool, it is not feasible to expect design engineers to perform a Monte Carlo simulation for joint on each project. The research team calculated joint width tolerances for typical joints in prefabricated bridge structures using Monte Carlo simulations using the proposed element and erection tolerances. These tolerances are included in tabular form in the proposed guideline.

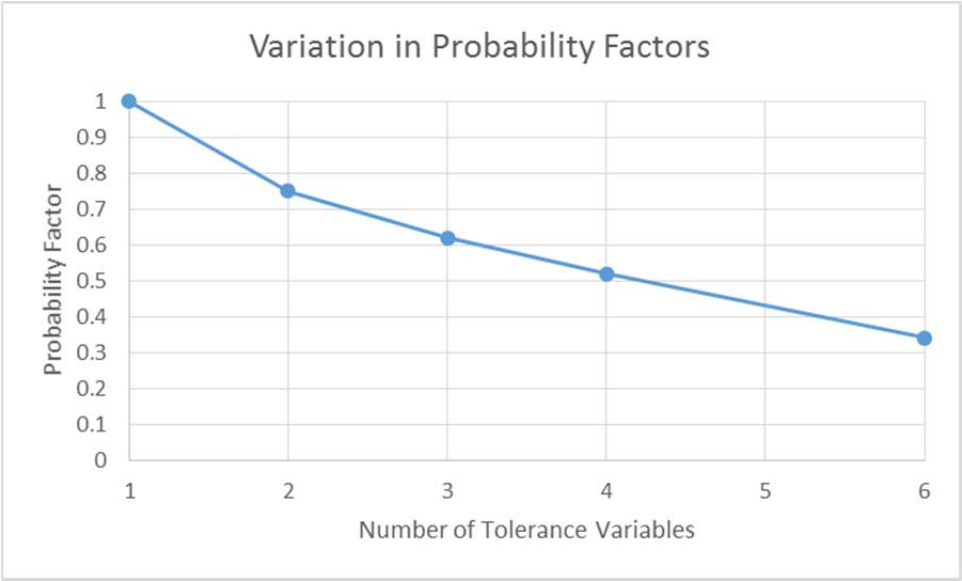
The results of the tolerance questionnaire indicate that there is general consensus with regard to element and erection tolerances; however the results were not unanimous. The team further investigated an approach to calculate joint width tolerances using owner/designer modified element and erection tolerances, but without the use of Monte Carlo simulations. An approach was developed that made use of the owner/designer proposed maximum element and erection tolerances combined with a factor to account for the probability of occurrence of the tolerances. The theory is that an owner/designer could enter the adjusted maximum tolerances for each variable and adjust the total by multiplying the result by a probability factor. Equation 4.6-4 is a modified version of Equation 4.6-1 with the addition of a probability factor.

$$T_{jw} = \pm\, \alpha\, [0.5(B_L + B_R + L_{HL} + L_{HR}) + \text{maximum of } (D_L + D_R),(H_L + H_R)] \qquad \text{(Eq. 4.6-4)}$$

Where:
α = *Probability factor*
All other factors being the same as in Equation 4.6-1

The probability factor is the ratio of the results of the Monte Carlo simulation divided by the result of Equation 4.6-1 (with maximum tolerances applied to each term). This approach accounts for the probability of occurrence of multiple variables without the use of Monte Carlo simulations. The proposed guideline includes these equations in the commentary for this purpose. The team investigated the accuracy of this method. A true Monte Carlo simulation was undertaken with modified tolerances and compared the results with the recommended simplified approach. The variation between these 2 methods was less than 1/8”, therefore the approach is feasible.

A review of the Monte Carlo simulations indicated a logical conclusion. Figure 4.6-2 is a plot of the probability factors for common joints versus the number of tolerance variables,



Note: There would be no reduction in probability if only one tolerance affected a joint; therefore the tolerance factor would be 1.0

Figure 4.6-2 Variation of Probability Factor Based on Number of Variables

The trend with the plot in Figure 4.6-2 is that the probability factor decreases as the number of variables increase. This is a logical conclusion. The probability of more variables maximizing at any joint is smaller since it would require more variables to be at or near their maximum, which would be less likely. For joints with just 2 variables, there is a higher likelihood that both would maximize at the same time.

Based on the success of this approach, joint width tolerances were calculated for each typical joint that is used in a typical prefabricated structure. Probability factors were also calculated for reinforcing penetrations in sockets, lapped splices, and sizes of pockets. The same process was used for this analysis.

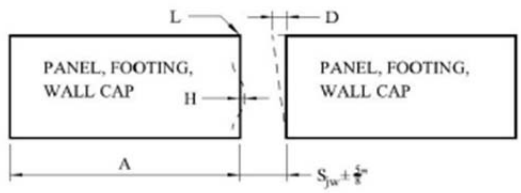
The results of the probability analyzes are included in Appendix B. Also included in the appendix is a description of the process used for the analysis. Figure 4.6-3 shows the results of one of the Monte Carlo simulations. This is for guideline Article 4.5.2.1, and specifically for Figure 4.5.2.1-5 in the guideline. In this case, 2 analyzes were run: one with erection tolerances included and one without. The reason for studying the elimination of erection tolerances was to attempt to keep tolerances reasonable by specifying that the panels could be reset if necessary. It was determined that the difference in tolerance between the 2 was small enough to include the erection tolerances. If a designer chose to use a narrower joint width, they could set those values to zero and use the process described in the commentary to calculate a new joint width specification.

Project:	NCHRP 12-98			Provision	4.5.2.1-5
		Prep'd by:	JLP	Date:	8/1/2016
Description:	App. Slab to App. Slab	Chk'd by:	MPC	Date:	9/1/2016

VERTICAL JOINT - PANEL TO PANEL JOINT TOLERANCE

CENTER OF ELEMENT LAYOUT METHOD

Applies to other vertical joints with same tolerances; which includes Pier Wall to Pier Wall, Backwall to Backwall, Cantilever Retaining Wall to Cantilever Retaining Wall, Integral Aubutment Wall to Integral Abutment Wall, Cantilever Abutment Wall to Cantilever Abutment Wall, and Footing to Footing.



ELEVATION

1. Input:

Label	Side	Type of Tolerance	Tolerance (in.)	Mean (in.)	STDV (in.)
A_L	Left panel	Width tolerance	0.25	0	0.125
A_R	Right panel	Width tolerance	0.25	0	0.125
H_L	Left panel	Edge flatness	0.25	0	0.125
H_R	Right panel	Edge flatness	0.25	0	0.125
D_L	Left panel	End squareness	0.5	0	0.250
D_R	Right panel	End squareness	0.5	0	0.250
L_{H_L}	Left panel	Horiz. Erection tolerance	0.5	0	0.250
L_{H_R}	Right panel	Horiz. Erection tolerance	0.5	0	0.250

Run	Min Joint Width Tolerance	Max Joint Width Tolerance	Description
1	1.25	0.25	Fabrication tolerances only (assume that panels can be re-erected to correct problems)
2	1.75	0.75	Fabrication & erection tolerances

2. Output:

	Min. Joint Width Tolerance		
Run	STD	TOL. (in.)	α
1	0.25	0.50	0.40
2	0.30	0.60	0.34

	Max. Joint Width Tolerance		
Run	STD	TOL. (in.)	α
1	0.09	0.18	0.71
2	0.20	0.39	0.52

Control Tolerance
MIN
MIN

3. Determine Tolerance Equation & Factor:

Conclusion: *A more conservative approach would be to use +/- Min. Width Joint Tolerances based on Min. Width Tolerance Factor and Min. Width Tolerance Equation.*

Run	Max. Tolerance (in.)	Modified Tolerance (in.)	α	Equation	Specified Joint Width Tol. (Solution to left eq. with max. tolerances)
1	1.25	0.50	0.40	$\alpha[0.5(A_L+A_R)+MAX(D_L+D_R,H_L+H_R)]$	0.498
2	1.75	0.60	0.34	$\alpha[0.5(A_L+A_R+L_{H\_L}+L_{H\_R})+MAX(D_L+D_R,H_L+H_R)]$	0.604

Figure 4.6-3 Example Monte Carlo Simulation Results

4.5 Conclusions

The attempts at collection of data from producers proved to be unsuccessful. Physical gathering of data by the research team was beyond the scope of this project and would be very time consuming involving travel to many states and physical measurements of thousands of elements. In many cases, the elements are stacked, making measurement nearly impossible.

The comparison of the analysis methods outlined in Section 4.2 with the PCI Tolerance Manual demonstrates the reality that the PCI Manual is a reasonable document to use as a basis for tolerances for precast PBES. Based on this, the following approach was used for the development of element tolerance and erection tolerance guidelines:

a. Develop a PBES tolerance guideline using the best available information. All indications from producers and owners are that the PCI Tolerance Manual is the best source. It is based on years of experience fabricating precast elements (the tolerances that a plant can produce on a regular basis). The reality is that there is no significant difference between vertical construction product and bridge product when it comes to fabrication.
b. Supplement the PCI Tolerance Manual with recommendations and guidance for designers and contractors on ABC projects.

A method for developing joint width and connection tolerances was developed accounting for the multiple element and erection tolerances that can affect joint widths. Logical equations were developed to determine the tolerances that affect joints and connections. In order to account for the probability of a multi-variable equation, Monte Carlo simulations were run. These simulations account for the probability of the variables maximizing at any one time. The results are a probability distribution for the entire equations. Using the probability theory that 95% of the data will reside within 2 standard deviations of the mean, probability factors were derived that can be applied to an equation that contains the maximum tolerance for each variable.

CHAPTER 5

Investigation of Dynamics for SPMT Bridge Systems

5.1 Introduction

There have been over 100 SPMT bridge moves completed in the United States in the last 10 years. The process of lifting, transporting and setting of bridge systems generally falls within the realm of temporary works and contractor means and methods. The design engineer has some responsibility to ensure that the bridge can be lifted in a reasonable manner at reasonable support locations, and that there is a reasonable travel path available for the movement of the bridge. The contractor is responsible for a number of things including the design of the falsework, the SPMT layout, and the travel path.

The issue of allowable twist during the bridge move has been somewhat variable. Some design engineers analyze the bridge for allowable twist, while others put this burden on the contractor. It is the opinion of the authors that the design engineer should determine the allowable twist that the structure can accommodate without damage based on the support conditions indicated on the plans. The designer has intimate knowledge of the structural system and its ability to twist and is; therefore, the more appropriate resource to determine allowable twist. The designer also should have control over limiting stresses in order to ensure minimal or no damage to the structure during the bridge move. Project specifications could include provisions for the contractor to perform analysis if he/she elects to alter the support points.

The process of lifting and moving a bridge is a dynamic event. Simple physics would dictate that the process of lifting, starting, stopping, and traversing uneven terrain will result in vertical and lateral accelerations and decelerations. Some might assert that these events are quasi-static; however, experience of the authors indicates that there are significant dynamic effects imparted on the bridge and falsework during the moves.

To date, large scale bridge system moves rely heavily on the expertise of the heavy lift company to determine the dynamic effects on the bridge system and temporary works during the actual move. A number of bridges have been moved with strain gages installed on the bridge members in order to measure strain and stress. These gages have measured stresses induced by vertical “bouncing” and twisting of the bridge during the move. There has been little or no investigation of the horizontal dynamic forces imparted on the structure during the moves. Horizontal forces have little effect on most bridge structures; however, they can have a dramatic effect on the temporary framework supporting the bridge. Horizontal dynamic effects are induced into the system during start-ups and stops of the transporters. CME Associates has designed several heavy lift systems. The magnitude of the lateral dynamic forces used for the framework design is typically based on rule-of-thumb experience of the heavy lift contractor. With more and more companies entering the market, the level of experience will inevitably be varied. This research was undertaken to formally determine the lateral dynamic effects using experimentation, and recommend specific design criteria for the temporary works. These set

criteria will dramatically improve safety for all heavy lift contractors by establishing the same standard regarding these horizontal forces.

Field instrumentation of an actual bridge move might seem like an appropriate method for gathering data for this research. Two members of the project team, CME Associates and USU, have completed a study of field data from a number of bridges installed in Utah with SPMTs (Rosvall, Et al (2010)). The data was gathered by the contractor prior to the start of the study; therefore there was no opportunity for controlling the type of data that was gathered. Still, the study was useful and resulted in preliminary recommendations for incorporation of vertical dynamic effect specifications into the Utah SPMT manual. This was done by computing the change in stress at high stress locations. The result was expressed as a percentage of the dead load stress.

Field data of actual bridge moves is valuable, but only to a degree. On most projects, operators take great care to start and stop as slow as possible, which most likely represents a lower bound on the potential forces. Design specifications need to be based on a specific limit state. It is not safe to take an actual bridge system and test for worst case scenarios. Based on this, the project team chose to measure accelerations on an SPMT loaded with simple weights such as crane counterweights. Under this scenario, the transporter can be safely started and stopped at maximum speed to determine the upper bound on design loadings.

Another reason to not use an actual bridge is to limit the secondary effects of the vibration of the falsework and bridge system. The premise of this research was to treat an SPMT bridge installation in a similar fashion as a seismic event. A structure being transported on an SPMT is subjected to lateral and vertical accelerations and decelerations. The accelerations at the top of the transporter are akin to ground level accelerations for a bridge in a permanent location. Measured base accelerations can be used to develop a response spectrum plot that can account for the effect of the stiffness of the bridge system in a design procedure. Once a response spectrum is developed, a design procedure can be developed that is similar to seismic design provisions.

5.2 SPMT Dynamic Testing

In order to establish an upper bound for accelerations, the team used an SPMT loaded with varying amounts of crane counterweights that was put through a series of typical bridge maneuvers all at maximum safe speeds. Simple accelerometers were placed on the transporter that can measure horizontal and vertical accelerations, with the goal of determining the maximum acceleration and deceleration of a loaded SPMT.

It is not possible to accommodate damping of the falsework above the transporter because each project uses different falsework. In an effort to study potential damping of the system, accelerometers were also being placed on top of the weights to see if there is any change.

An SPMT was loaded with varying amounts of load (such as crane counterweights).

- Empty
- 25% capacity
- 50% capacity

The reason for varying the weight was based on the fact that acceleration and braking on an SPMT is a function of the power in the system. With constant acceleration and braking power, the accelerations and

decelerations would in theory be larger for lightly loaded SPMTs. The goal was to capture this effect in the testing.

5.2.1 Sensor Placement

Nine accelerometers were installed in 4 different locations on the SPMT platform and the payload. Each accelerometer was mounted magnetically in one of 3 principle directions: Longitudinal, Transverse and Vertical. Positive longitudinal is defined as the forward direction of the SPMT. Positive transverse is defined as perpendicular to the longitudinal direction on the platform plane, using the right hand rule with positive vertical defined as opposite of gravity (i.e. up). The direction of each accelerometer is shown in Table 5.2.1-1. Specifications for the accelerometers are shown in Table 5.2.1-2.

Positon	Designation	Direction
Front left on platform	A2130	Longitudinal
	A2131	Transverse
	A2132	Vertical
Back-right on platform	A2133	Longitudinal
	A2089	Transverse
	A2090	Vertical
On the weights	A1036	Longitudinal
	A1035	Transverse
	A2087	Vertical

Table 5.2.1-1 Accelerometer Placement on the SPMT

A cluster of 3 accelerometers, measuring the 3 principle directions, were attached to the front left side, the back left side and on top of the metal weights on the platform. The accelerometer placements are shown in Figure 5.2.1-3. Three nodes were used to connect the accelerometers to the base station and laptop.

Name	Range	Calibration Factor
A1035	±2g	1.00281 g/V_{OUTPUT}
A1036	±2g	1.00648 g/V_{OUTPUT}
A2087	±5g	325.733 $g/V_{OUTPUT}/V_{EXCITATION}$
A2088	±5g	326.797 $g/V_{OUTPUT}/V_{EXCITATION}$
A2089	±5g	332.266 $g/V_{OUTPUT}/V_{EXCITATION}$
A2090	±5g	319.489 $g/V_{OUTPUT}/V_{EXCITATION}$
A2130	±5g	2.516 g/V_{OUTPUT}
A2131	±5g	2.516 g/V_{OUTPUT}
A2132	±5g	2.516 g/V_{OUTPUT}
A2133	±5g	2.513 g/V_{OUTPUT}

Table 5.2.1-2 Accelerometer Specifications

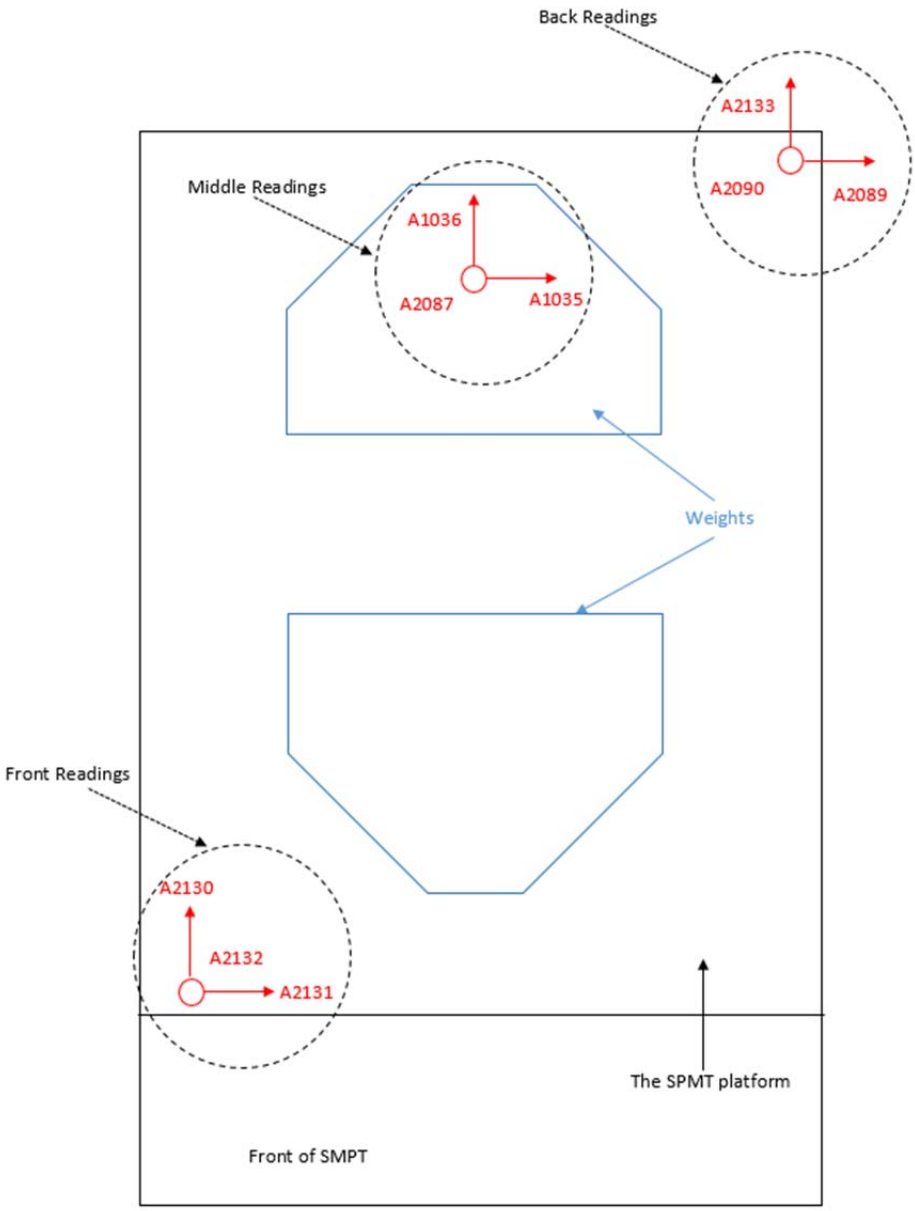


Figure 5.2.1-3 Schematic Plan of Accelerometer Placement

A payload consisting of metal crane weights was secured to the platform. Three load cases defined as heavy, medium and light, with 100 metric Tons, 50 metric Tons and 15 metric Tons of payload, respectively. The 100 Ton weight represents approximately 50% of the capacity of the SPMT and the 50 Ton weight represents approximately 25% of the capacity. Higher load cases were considered, however there was concern regarding the safety of the operation and potential to damage the machine. By measuring 3 different load cases, the team could develop a mathematical relationship between payload and acceleration.

5.2.2 Motion Cases

The following scenarios, defined as Motion Cases, were defined and investigated to cover all possible extreme events during the bridge transport:

- "Start and Stop": A sudden brake or a rapid take off can cause considerable longitudinal accelerations. This scenario is considered the most common and can occur several times during one carriage, although typically not to the extreme.
- "Up and Down": Bouncing of the vehicle can cause significant acceleration in vertical direction. This may occur during raising or lowering of the cargo into place.
- "90 Degree Turn": Turning while moving a bridge deck on the road can create both longitudinal and transverse accelerations.
- "Long Run": During a transport, it is possible to encounter uneven ground or the weight of the loaded SPMT can create uneven ground in poorly prepared roadbed. This motion case is characterized by a long run of road unevenness with locations of well and poorly compacted soil, causing significant vertical accelerations.
- "Rotation": SPMTs can rotate 360 degrees causing accelerations in horizontal directions.

For each load case, defined in the following section, all of the 5 motion cases were performed. During the test, the operator of the SPMT was asked to operate the machine at the maximum safe speeds.

5.3 Load Cases

As previously described, 3 load case tests were completed in order to capture the effect of the transporter weight on the dynamic behavior. The following sections contain the results of each load case

5.3.1 Heavy Load Case Tests (100 Metric Tons)

The test SPMT was loaded with 2 stacks of crane counter weights: 9 steel plates on one side and 8 on the other side. Each individual plate weighed 5 Tons. Additionally, a steel platform on which the weights were placed weighed 15 Tons, thus the total load on the SPMT was 100 Tons. Figure 5.3.1-1 shows the loaded SPMT used in the test.

As described in Section 5.2.1, a cluster of 3 accelerometers was installed on the top of the front stack at the elevation of 108 in. above the SPMT's platform (deck).

Figure 5.3.1-1 SPMT Set-Up for Heavy Load Case

The heavy load case test included the following motion cases:

- **SPMT1: Test, checking the sensors**. Note the results of this will not be reported for brevity.
- **SPMT2: Up and down**. Two up-down cycles for this motion case were carried out.
- **SPMT3: Misfire.** The vehicle started before the running of data acquisition system. Note the results of this will not be reported for brevity.
- **SPMT4: Start and stop**. This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT5: Start and stop**. This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT6: Start and stop**. This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT7: Start and stop**. This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT8: 90 degree turn**. The SPMT was turned for 90 degrees to the South without stopping.
- **SPMT9: Long run**. The SPMT moved forward at a constant fast walking pace while on uneven ground.
- **SPMT10: Long run**. The SPMT moved backward at a constant fast walking pace while on uneven ground.
- **SPMT 11: Rotation**. The SPMT was rotated 360 degrees clockwise.

Figure 5.3.1-2 is a schematic map for the motions in this load case.

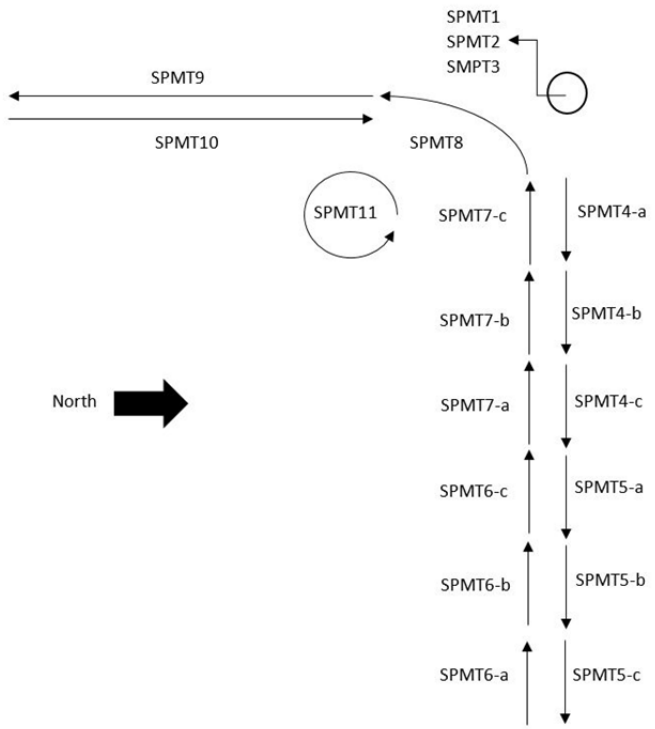


Figure 5.3.1-2 Schematic Map of Motion Cases for Heavy Load Case

5.3.2 Medium Load Case Tests (50 Metric Tons)

The vehicle was loaded with 2 stacks of crane counter weights: 4 steel plates on one side and 4 on the other side. Additionally, a steel platform on which the weights were placed weighed 15 Tons, thus the total load on the SPMT was 50 Tons.

As described in Section 5.2.1, a cluster of 3 accelerometers was installed on the top of the front stack at the elevation of 51 in. from the SPMT's platform bottom.

The medium load case test included the following motion cases:

- **SPMT12: Test, checking the Sensors.** Note the results of this will not be reported for brevity.
- **SPMT13: Moving back into original position.** Note the results of this will not be reported for brevity.
- **SMPT14: Start and stop.** This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT15: Start and stop.** This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT16: Start and stop.** This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT17: Start and stop.** This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT18: 90 degree turn.** The SPMT was turned for 90 degrees to the South without stopping.
- **SPMT19: Long run.** The SPMT moved forward at a constant fast walking pace while on uneven ground.
- **SPMT20: Long run.** The SPMT moved backward at a constant fast walking pace while on uneven ground.
- **SPMT21: Rotation.** The SPMT was rotated 360 degrees counter clockwise.
- **SPMT22: Up and down.** Two up-down cycles for this motion case were carried out.

Figure 5.3.2-1 is a schematic map for the motions in this load case.

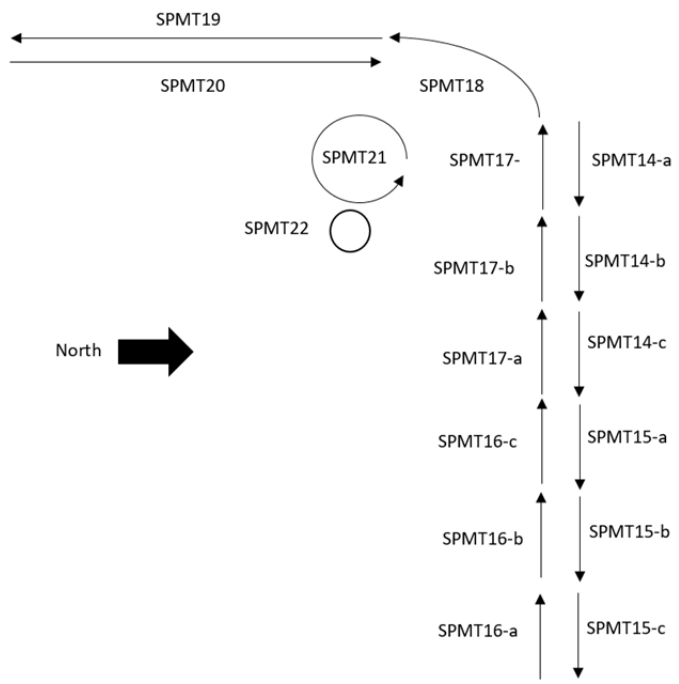


Figure 5.3.2-1 Schematic Map of Motion Cases for Medium Load Case

5.3.3 Light Load Case Tests (15 Metric Tons)

All the steel weights were unloaded and only the platform of SPMT remained. The total carrying weight by the SPMT was 15 Tons in this case.

The light load case test included the following motion cases:

- **SPMT23: Test, checking the sensors.** Note the results of this will not be reported for brevity.
- **SPMT24: Up and down.** Two up-down cycles for this motion case were carried out.
- **SPMT25: Moving back into original position.** Note the results of this will not be reported for brevity.
- **SMPT26: Start and stop.** This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT27: Start and stop.** This test was conducted in backward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT28: Start and stop.** This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SMPT29: Start and stop.** This test was conducted in forward direction. During the test the SPMT started and stopped 3 times (a, b and c).
- **SPMT30: 90 degree turn.** The SPMT was turned for 90 degrees to the South without stopping.
- **SPMT31: Long run.** The SPMT moved forward at a constant fast walking pace while on uneven ground.
- **SPMT32: Long run.** The SPMT moved backward at a constant fast walking pace while on uneven ground.
- **SPMT32: Rotation.** The SPMT was rotated 360 degrees counter clockwise.

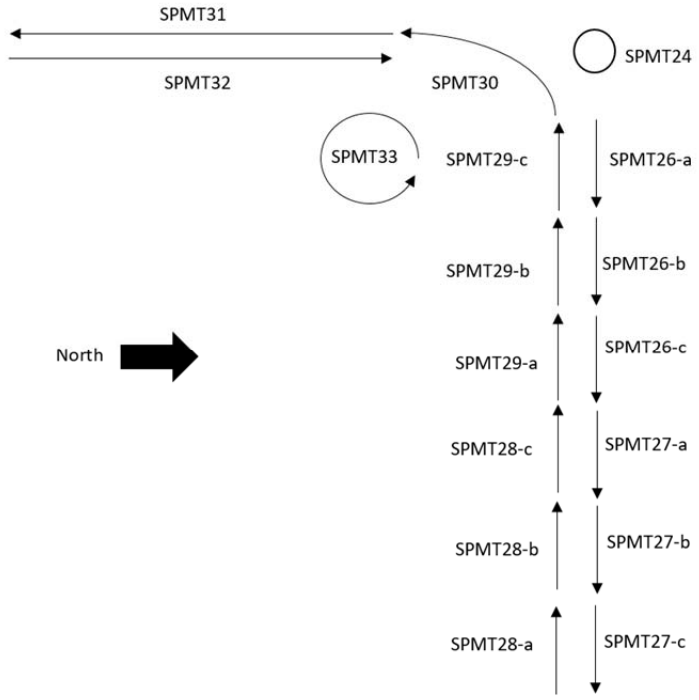


Figure 5.3.3-1 Schematic Map of Motion Cases for Light Load Case

Figure 5.3.3-1 is a schematic map for the motions in this load case.

5.4 Experimental Results

The sensors were divided into 3 clusters based on their location. Each cluster consists of 3 sensors, one for each principle direction. The clusters shown in Figure 5.2.1-3 are grouped together as follows:

- **Cluster 1 (Front Readings):** including 3 sensors in 3 defined directions located in front of the SPMT (A2133-L, A2089-T, and A2090-V).
- **Cluster 2 (Middle Readings):** including 3 sensors in 3 defined directions located in the middle of the SPMT i.e. on top of the weights (A1036-L, A1035-T, and A2087-V).
- **Cluster 3 (Back Readings):** including 3 sensors in 3 defined directions located in back of the SPMT (A2130-L, A2131-T, and A2132-V).

All accelerometers were sampled at a rate of 100 Hz.

5.4.1 Measured Peak Platform Accelerations

The peak platform accelerations (PPA) for each cluster, load case and motion case are presented in Table 5.4.1-1 through Table 5.4.1-3 for Cluster 1 through Cluster 3, respectively. All time history plots for the acceleration data recorded for each motion and load case are presented in Section 5.8.

Motion Case	Principle Direction	15 Tons	50 Tons	100 Tons
Start and Stop	Longitudinal	0.34g	0.23g	0.25g
	Vertical	0.57g	0.22g	0.13g
	Transverse	0.10g	0.08g	0.07g
Long Run	Longitudinal	0.11g	0.15g	0.06g
	Vertical	0.57g	0.12g	0.08g
	Transverse	0.14g	0.10g	0.08g
Up and Down	Longitudinal	0.06g	0.04g	0.04g
	Vertical	0.05g	0.06g	0.07g
	Transverse	0.06g	0.03g	0.05g
90 Degree Turn	Longitudinal	0.05g	0.03g	0.04g
	Vertical	0.05g	0.04g	0.07g
	Transverse	0.07g	0.06g	0.04g
Rotation	Longitudinal	0.04g	0.02g	0.03g
	Vertical	0.08g	0.04g	0.04g
	Transverse	0.12g	0.11g	0.07g

Table 5.4.1-1 Maximum Accelerations for Cluster 1 (Top of SPMT Platform)

Motion Case	Principle Direction	15 Tons	50 Tons	100 Tons
Start and Stop	Longitudinal	0.34g	0.29g	0.28g
	Vertical	0.59g	0.14g	0.10g
	Transverse	0.09g	0.08g	0.07g
Long Run	Longitudinal	0.11g	0.16g	0.06g
	Vertical	0.34g	0.12g	0.07g
	Transverse	0.13g	0.12g	0.09g
Up and Down	Longitudinal	0.06g	0.05g	0.05g
	Vertical	0.04g	0.05g	0.05g
	Transverse	0.05g	0.02g	0.05g
90 Degree Turn	Longitudinal	0.05g	0.08g	0.04g
	Vertical	0.03g	0.05g	0.03g
	Transverse	0.06g	0.06g	0.05g
Rotation	Longitudinal	0.03g	0.03g	0.03g
	Vertical	0.04g	0.03g	0.03g
	Transverse	0.10g	0.09g	0.06g

Table 5.4.1-2 Maximum Accelerations for Cluster 2 (Top of Payload)

Motion Case	Principle Direction	15 Tons	50 Tons	100 Tons
Start and Stop	Longitudinal	0.33g	0.23g	0.25g
	Vertical	0.18g	0.12g	0.18g
	Transverse	0.09g	0.07g	0.07g
Long Run	Longitudinal	0.11g	0.16g	0.06g
	Vertical	0.25g	0.13g	0.13g
	Transverse	0.12g	0.08g	0.07g
Up and Down	Longitudinal	0.06g	0.04g	0.04g
	Vertical	0.08g	0.06g	0.04g
	Transverse	0.05g	0.02g	0.04g
90 Degree Turn	Longitudinal	0.05g	0.03g	0.03g
	Vertical	0.05g	0.05g	0.05g
	Transverse	0.05g	0.05g	0.04g
Rotation	Longitudinal	0.06g	0.03g	0.03g
	Vertical	0.06g	0.03g	0.04g
	Transverse	0.11g	0.06g	0.03g

Table 5.4.1-3: Maximum Accelerations for Cluster 3 (top of SPMT platform)

Comparing the values between Cluster 1 through Cluster 3, the maximum accelerations are very similar between motion and load cases. The largest differences have come between the transverse directions for the rotation load case which is likely affected by the evenness of the driving surface, as discussed in more detail below. A notable exception is the vertical acceleration during the Long Run for the 15 T load case, in which Cluster 1 reported 0.57g compared to 0.25g. This is also exhibited when comparing the Start and Stop Motion Case for the 15 T load case (compare 0.57g to 0.18 in Table 5.4.1-1 and 5.4.1-3). This difference is likely related to the center of rotation of the SPMT system, which includes the 30 ft long power pack which houses the hydraulic system. Realistically, due to measurement and testing error, most readings between Cluster 1, 2 and Cluster 3 provide identical results.

The maximum longitudinal acceleration occurred during the Start and Stop Motion Case (SPMT 7) with the maximum value of 0.28g. The maximum vertical acceleration occurred during the Start and Stop Motion Case (SPMT 7) with 0.18g acceleration. The maximum transverse acceleration for the 100 T load case was happened during the Long Run Motion Case (SPMT 9) with 0.08g acceleration. For the 100 T load case, the Start and Stop Motion Case had the controlling acceleration in longitudinal and vertical directions. The maximum transverse acceleration for the 100 T load case occurred during the Long Run Motion Case (SPMT 10) with 0.07g acceleration.

For the 50 T load case, the maximum longitudinal acceleration occurred during the Start and Stop Motion Case (SPMT 17) with a maximum value of 0.23g. The maximum vertical acceleration occurred during the Start and Stop Motion Case (SPMT 17) with 0.21g acceleration. However, the maximum transverse acceleration for this load case was happened during the Rotation Motion Case (SPMT 21) with 0.11g acceleration. Similar to the heavy load case, the controlling motion case for longitudinal and vertical accelerations was Start and Stop, but the Rotation Motion Case controlled the transverse acceleration.

For the 15 T load case, the maximum longitudinal acceleration occurred during the Start and Stop Motion Case (SPMT 29) with the maximum value of 0.34g while the maximum vertical acceleration also occurred during the Start and Stop Motion Case (SPMT 28) with a value of 0.59g. The maximum transverse acceleration for 15 T load case was recorded during the Long Run Motion Case (SPMT 31) with 0.14g acceleration. The controlling motion cases for longitudinal, vertical and transverse accelerations recorded during Start and Stop, Start and Stop, and Long Run Motion Cases, respectively.

Figure 5.4.1-3 through Figure 5.4.1-5 represent the maximum longitudinal, Transverse and Vertical acceleration time histories. Figure 5.4.1-2 presents the maximum longitudinal acceleration time history from SPMT29. This time history is characterized by the 3 main stop and start maneuvers where each stop and start has a significant positive and negative pulse. The negative pulse is always larger than the positive pulse in which the vehicle is stopping. This indicates that an emergency stop for SPMT is more critical for design than an abrupt start. Figure 5.4.1-3 presents the acceleration time history for SPMT31 which is the maximum transverse time history and occurred during a Long Run Motion Case. The transverse accelerations are pulses of very short duration as opposed to the large impulses noticed during the longitudinal accelerations of the Start and Stop Motion Case. Figure 5.4.1-4 presents the acceleration time history for SPMT28 which is the maximum vertical time history that occurred during a Start and Stop Motion Case. The time histories for maximum vertical accelerations are significantly different, with the Long Run Motion Case exhibiting a longer and more sustained vertical shaking than the relatively short duration Start and Stop accelerations.

The transverse accelerations were significantly affected by the gravity reference of the sensors. If the SPMT platform tilted to the side slowly due to uneven ground or natural changes in elevation the sensor

would register a change in acceleration, even though this change is pseudo static. For instance: the maximum transverse acceleration occurred during the Long Run Motion Case and, as shown in Figure 5.4.1-1, due to unevenness of the road, the accelerometer readings were slowly skewed, with maximum dynamic vibration as indicated by the arrow. Therefore, the actual value of transverse acceleration that the falsework or carried structure would experience dynamically is less than 0.08g (approximately 0.04g) and occurred over a very short time period, which is likely to have little effect on the falsework or bridge structure. This phenomenon is consistent across all transverse accelerations in all motion and load cases.

Maximum vertical accelerations were considerably higher in the Start and Stop Motion Case than any other (e.g., 0.59g in Table 5.4.1-2). However, this is somewhat misleading because of the rotation of the platform during the starting and stopping maneuver. During a stop, vertical accelerations are generated by the rotation of the platform about some point between the center of the platform and the back axle, due to the heavy power pack attachment. During a large bridge move it is assumed that more than one SPMT will be linked, whether directly attached or through the attachment of the falsework and bridge assembly. By linking the SPMTs this rotation is assumed to be restrained significantly so only the horizontal readings from the Start and Stop Motion Case will be considered for design values (elaborated in the following section).

Maximum vertical accelerations during the Long Run Motion Case were caused by the velocity of the SPMT navigating uneven ground. For this reason, vertical accelerations from the Long Run Motion Case are considered critical to design. Maximum longitudinal accelerations were significant in the Long Run Motion Case, but rarely controlling simply based on the maximum measured accelerations. The maximum vertical accelerations during the Long Run Motion Case are 0.57g, 0.13g and 0.13g, for 15 T, 50T and 100T respectively.

The Motion Cases of Up and Down, 90 Degree Turn and Rotation did not exhibit the most severe longitudinal, Transverse or Vertical accelerations when compared to the Start and Stop and Long Run Motion Cases. For this reason, the results from the Start and Stop and Long Run Motion Cases are recommended for development of the design specifications.

In general, maximum accelerations occurred during the 15 T load case. This is likely due to the higher velocities achievable with lower loads, the ability of the SPMT to stop and change in center of rotation of the SPMT and power pack. The operator also expressed some concern at breaking the SPMT hydraulics at higher loads which may have affected the maximum accelerations at the 100 T load.

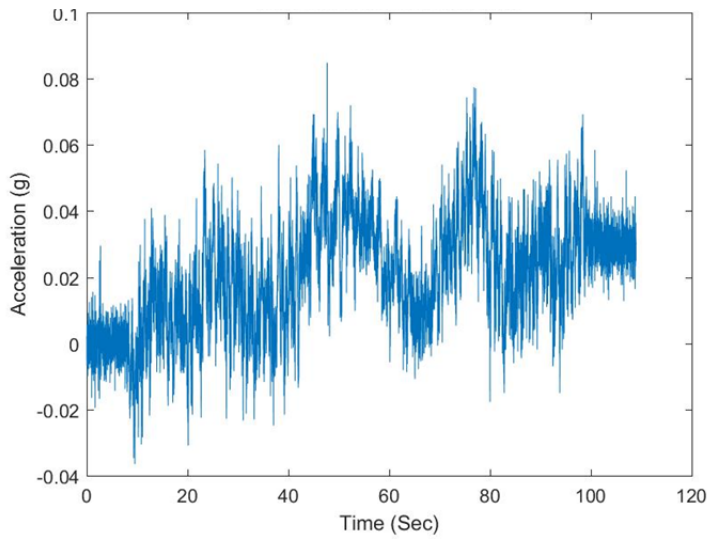


Figure 5.4.1-1 Time History for Maximum Transverse Acceleration for Cluster 1 with 100 T Load Case

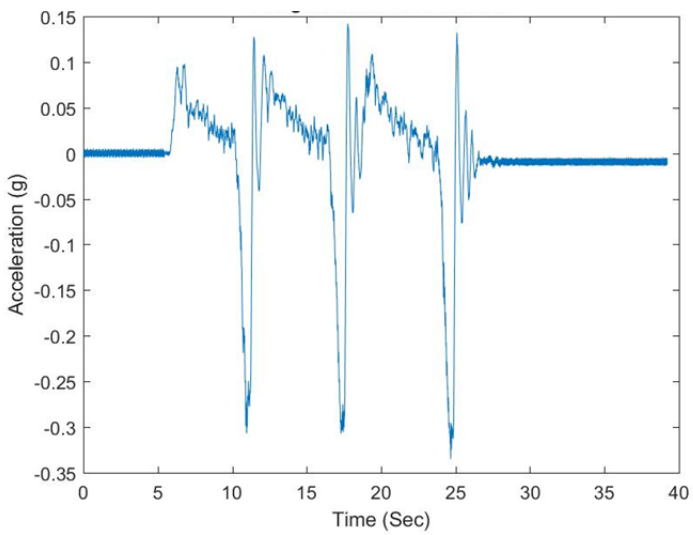


Figure 5.4.1-2 SPMT29 Acceleration Time History, Cluster 1 Longitudinal Acceleration Absolute Maximum

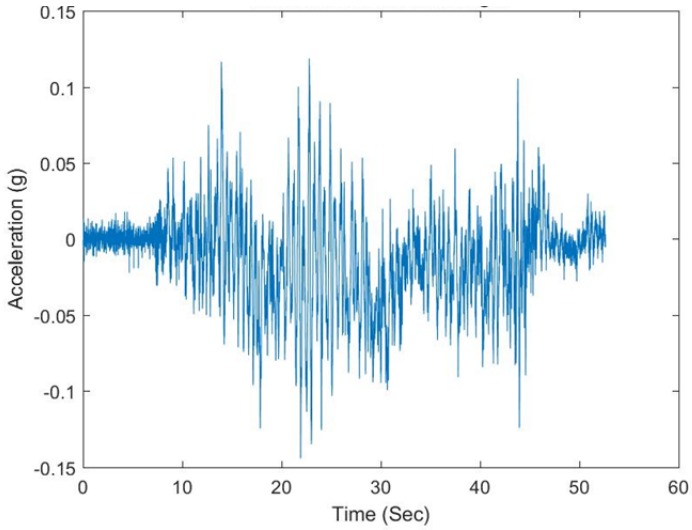


Figure 5.4.1-3 SPMT31 Acceleration Time History, Cluster 1 Transverse Acceleration Absolute Maximum

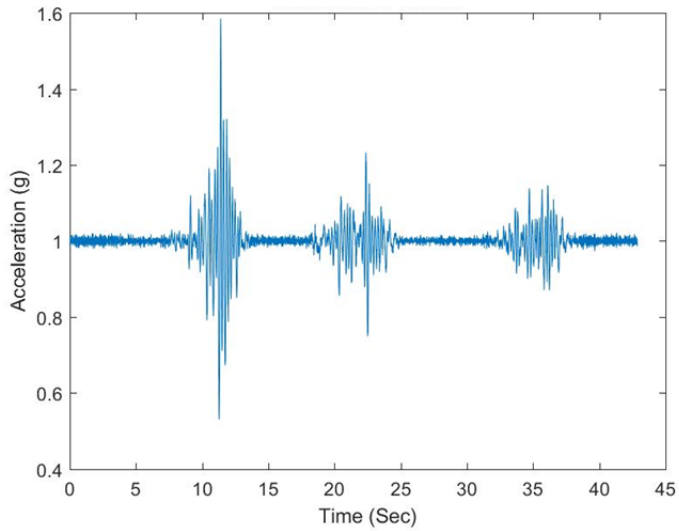


Figure 5.4.1-4 SPMT28 Acceleration Time History, Cluster 2 Maximum Vertical Acceleration from Start and Stop Motion Case

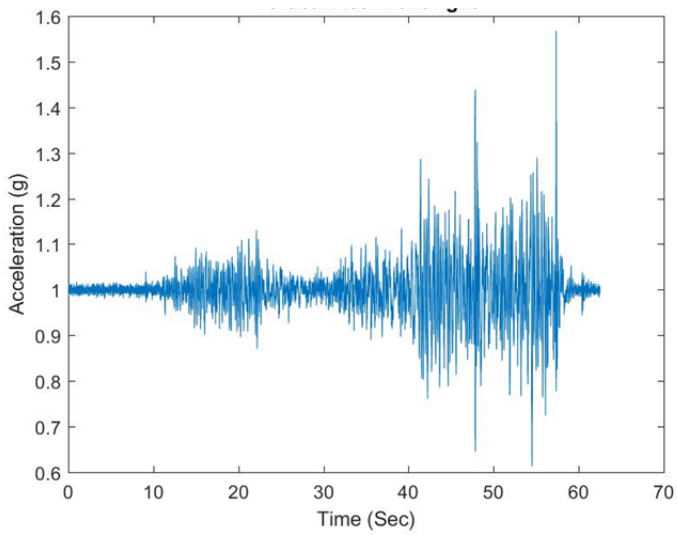


Figure 5.4.1-5 SPMT32 Acceleration Time History, Cluster 1 Maximum Vertical Acceleration from Long Run Motion Case

5.5 Response Spectra

For design purposes, the time history accelerations presented in the previous section and Section 5.8 are difficult to use in a design guideline. For this reason, response spectra are generated in this section using single degree of freedom assumptions similar to those already used for seismic design in the AASHTO LRFD Bridge Design Specifications. The assumed single degree of freedom system uses the platform accelerations as a dynamic problem with base excitation (Figure 5.5-2). Therefore, the following differential equation needs to be solved to get the dynamic response of the system (falsework and bridge).

$$\ddot{u} + 2\zeta\omega\dot{u} + \omega^2 u = -\ddot{u}_p \qquad \text{(Eq. 5.5-1)}$$

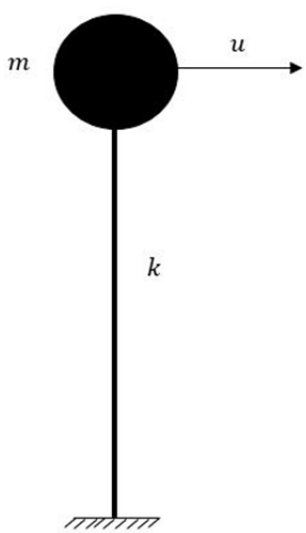


Figure 5.5-2 SDOF System

Where $u, \dot{u}, \ddot{u},$ and $\ddot{u}_p$ are the displacement of the system, velocity of the system, acceleration of the system, and the platform acceleration, respectively, at any time interval of the experiment. The value ζ and ω are the damping ratio of the system and the natural frequency of the system, respectively.

Platform accelerations ($\ddot{u}_p$) are the measured time history graphs for each motion case in each load case. To solve equation (5.5-1), the Newmark-beta method was used. The approach in this solution was based on the constant average acceleration assumption at each time interval of the time history graphs (0.01 s).

Damping ratio ζ and the natural frequency ω are assumed deterministic. For SPMT and the false work, the damping ratio $\zeta = 0.02$ is selected. The period of the system ($2\pi/\omega$) is analyzed between 0.02 second and 4 second increasing with 0.01 second steps. The pseudo acceleration response history of each time history for each period is then calculated and the maximum value of pseudo acceleration in each period was determined and plotted versus period to obtain the response spectra graphs presented in the following section.

5.5.1 Spectrum for Vertical Motions

The Long Run Motion Cases were selected to obtain the spectrum graphs in vertical direction as they were deemed most critical for vertical vibrations. The Long Run Motion Cases were carried out twice for each load case with 3 vertically oriented accelerometers, therefore 6 spectrum graphs were obtained.

The response spectra accelerations are significantly affected by the operator during each incident, for instance, the breaking and turning speeds, as well as the terrain. Thus, a composite mean acceleration spectra and mean value plus one standard deviation of the spectra have also been calculated. In the following response spectra, the mean graph (labeled as μ) provides a proper estimation of the imposed accelerations to the SPMT during the transportation. For spectrum graphs to cover most of the Long Run Motion Cases, the mean plus one standard deviation graph (labeled as μ+σ) is plotted.

Figure 5.5.1-1 presents all vertical spectrum graphs for the 15 T load case with grey dashed lines. The μ graph and the μ+σ composites are also presented with continuous and dashed lines, respectively. The maximum pseudo acceleration at the minimum period (0.01s) is consistent with the PPA at each load case. The estimated PPA for 15 T load case in the μ+σ composite is approximately 0.85g. As shown in Figure 5.5.1-1, the period at the maximum pseudo acceleration is at a period of 0.26 s with the maximum vertical acceleration of 2.3g. Also after the period of 1.5 second, after which the graph levels out considerably. The maximum acceleration and the period of the maximum acceleration are thought to be highly dependent on the roughness of the terrain and behavior of the operator, and therefore, highly variable.

Figure 5.5.1-2 presents all vertical spectrum graphs for the 50 T load case with grey dashed lines. The μ graph and the μ+σ composites are also presented with continuous and dashed lines, respectively. The estimated PPA for 50 T load case in the μ+σ composite is approximately 0.19g. The maximum pseudo acceleration occurs at a period of 0.28 s with a pseudo acceleration of 1.07g. After this point, the graph begins to level off and is near zero after 2.5 s.

Figure 5.5.1-3 presents all vertical spectrum graphs for the 100 T load case. The μ graph and the μ+σ composites are also presented with continuous and dashed lines, respectively. The estimated PPA for the 100 T load case in the μ+σ composite is approximately 0.21g. The maximum pseudo acceleration occurs

at a period of 0.36 s with a pseudo acceleration of 0.62g. After this point, similar to Figure 5.5.1-1 and Figure 5.5.1-2, the pseudo acceleration drops to near zero after the period of 2.5 s.

Clearly, the maximum pseudo accelerations decrease significantly as the SPMT load increases. Furthermore, it seems as the payload increases, the period of the maximum pseudo acceleration response increases. This implies pseudo accelerations have a dependence on SPMT velocity and it was noticed that maximum attainable velocity during the Long Run Motion Case decreased with increased SPMT load because the terrain was essentially unchanged from iteration to iteration. Exact velocities were not measured, but maximum velocities were estimated at approximately 4 mph (fast walk) during the Long Run Motion Case, but slightly decreased between the 15 T to 50 T to 100 T Load Cases.

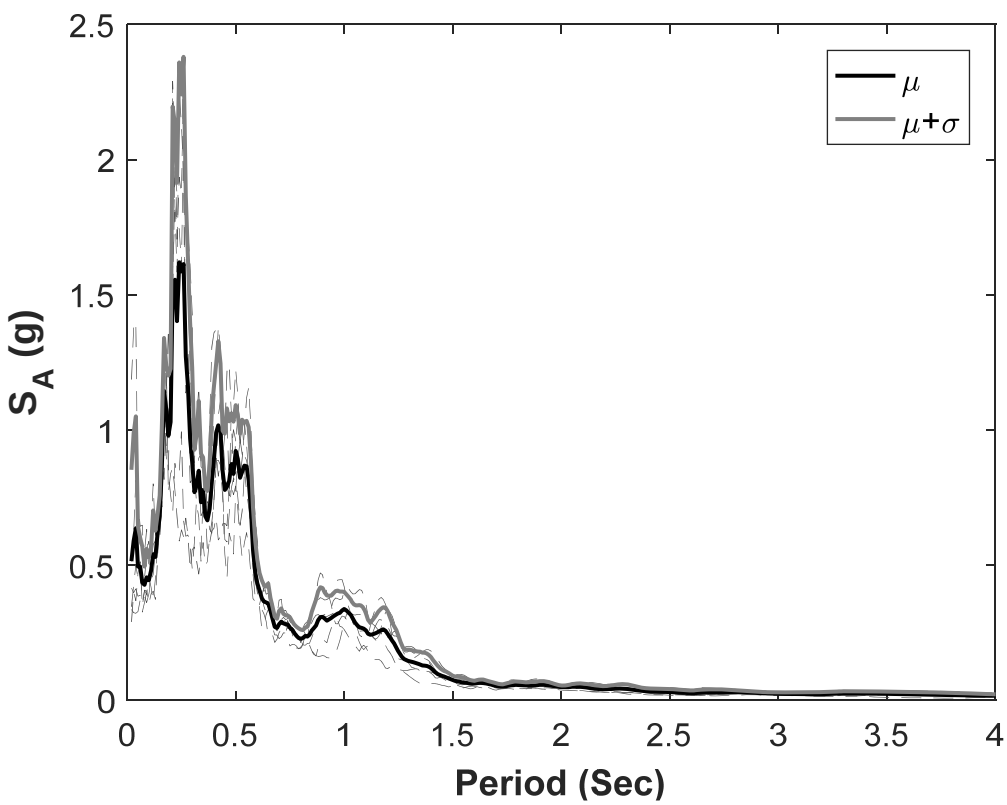


Figure 5.5.1-1 Response Spectra Mean and Mean Plus One Standard Deviation for Vertical Accelerations, 15T

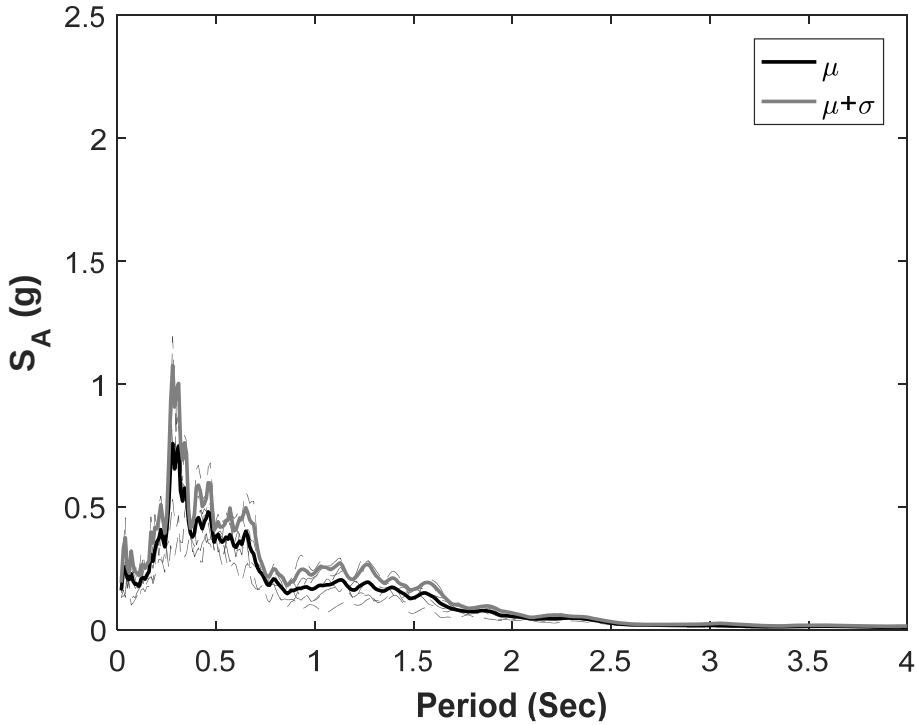


Figure 5.5.1-2 Response Spectra Mean and Mean Plus One Standard Deviation for Vertical Accelerations, 50 T

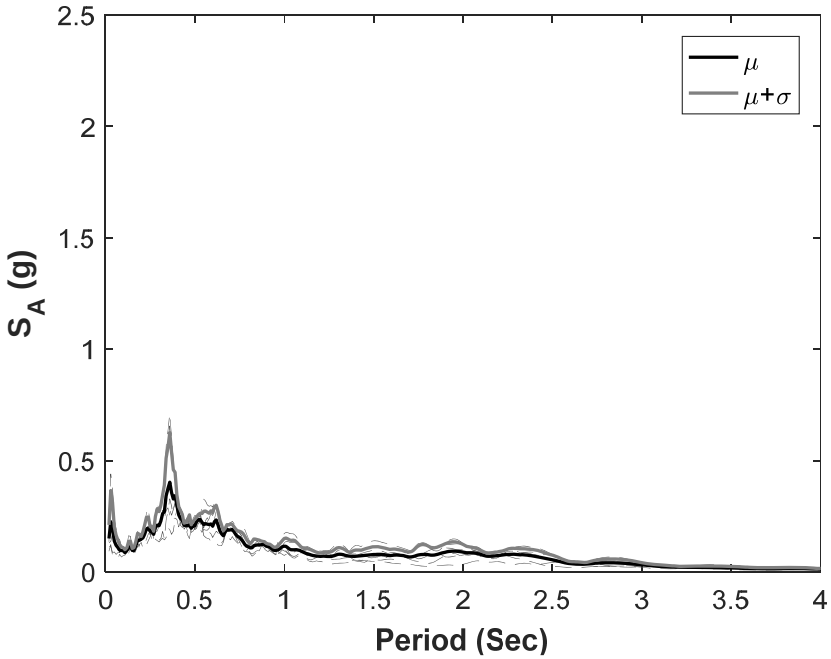


Figure 5.5.1-3 Response Spectra Mean and Mean Plus One Standard Deviation for Vertical Accelerations, 100 T

5.5.2 Spectrum for Horizontal Motions

The Start and Stop Motion Cases were selected to obtain the spectrum graphs in horizontal direction because they were deemed most critical for horizontal impact and vibrations. The Start and Stop Motion Cases included 3 instances of starts and stops (see Figure 5.4.1-2). To prevent interaction between starts and stops in the analysis, which would not mimic real life actions, the time histories were broken up and each Start and Stop instance was treated as a new time history to calculate the corresponding spectrum. The "Start and Stop" Motion Cases were carried out 4 times for each load case. There were 3 accelerometers measuring horizontal accelerations in each experiment therefore there were 36 total acceleration time histories for each load case, and therefore, 36 unique response spectra can be obtained.

As with the vertical acceleration time histories, the response spectra accelerations are significantly affected by the operator during each incident, who applied various degrees of breaking as evidenced by the time histories in Figure 5.4.1-2 and Section 5.8). To account for this variability, while remaining aware of realistic emergency stop scenarios, a composite μ spectra and μ+σ spectra have also been calculated.

Figure 5.5.2-1 presents the horizontal spectra for the 15 T load case. The μ spectra and μ+σ spectra are also presented with dark continuous and dashed lines, respectively. The estimated PPA for the 15 T load case in the μ+σ spectrum is approximately 0.34g. The period of the maximum pseudo acceleration is at a period of 0.57 s with an acceleration of 0.6g. After this peak, the resulting spectral accelerations decrease almost linearly, but not as dramatically as seen in previous graphs, indicating significant response across all structure periods considered. The maximum acceleration and the period of the maximum acceleration are highly dependent on the duration of the Start and Stop impulse, as well as the peak value. It is reasonable that a short duration pulse will excite stiffer structures more and a slower stop would excite a more flexible structure.

Figure 5.5.2-2 presents the horizontal spectra for the 50 T load case. The μ spectra and μ+σ spectra are also presented with dark continuous and dashed lines, respectively. The estimated PPA for the μ+σ spectrum is 0.24g. The period of the maximum pseudo acceleration is at a period of 0.71 s with a maximum horizontal pseudo acceleration of 0.47g. Similar to the 15 T spectra, after the maximum point, there is a nearly linear acceleration decrease, differing considerably from the vertical spectra in the previous section.

Figure 5.5.2-3 presents the horizontal spectra for the 100 T load case. The μ spectra and μ+σ spectra are also presented with dark continuous and dashed lines, respectively. The estimated PPA for the μ+σ spectrum is 0.18g, considerably lower than the 15 T and 50 T Load Cases. The maximum pseudo acceleration occurs at a period of 0.86 s with a pseudo acceleration of 0.32g, also considerably lower than the 15 T and 50 T Load Cases.

As with the vertical spectra, the maximum pseudo accelerations decrease significantly as the SPMT load increases. Furthermore, as the payload increases, the period of the maximum pseudo acceleration response increases. Because of the significant motion case differences, the horizontal accelerations are thought to be more of a function of the deceleration rather than maximum velocity. Consider that, if the operator imposes maximum breaking force, as in a worst case emergency stop, the deceleration rate will be constant (limited by the hydraulics, payload and operator capability) as the vehicle comes to a stop. It is clear from the observed variability in the Start and Stop Motion Case acceleration time histories that the operator, even when trying, cannot apply the same maximum deceleration. For this reason the μ+σ composite spectra are considered adequate for development of design spectra.

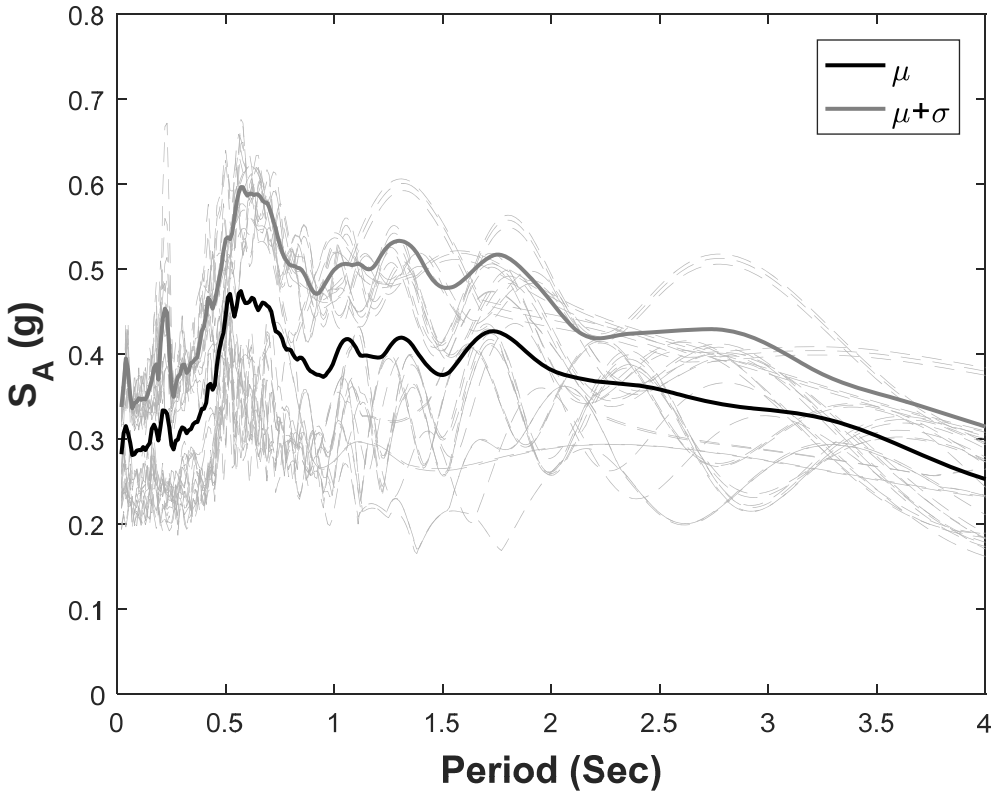


Figure 5.5.2-1 Response Spectra Mean and Mean Plus One Standard Deviation for Horizontal Accelerations, 15 T

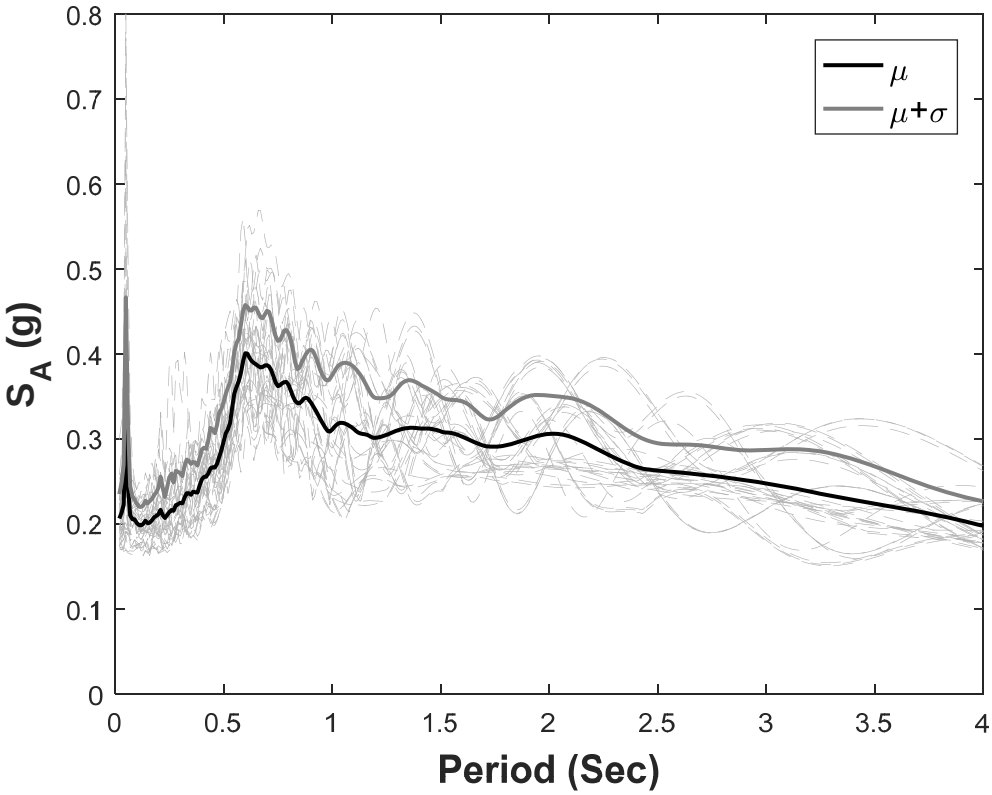


Figure 5.5.2-2 Response Spectra Mean and Mean plus One Standard Deviation for Horizontal Accelerations, 50 T

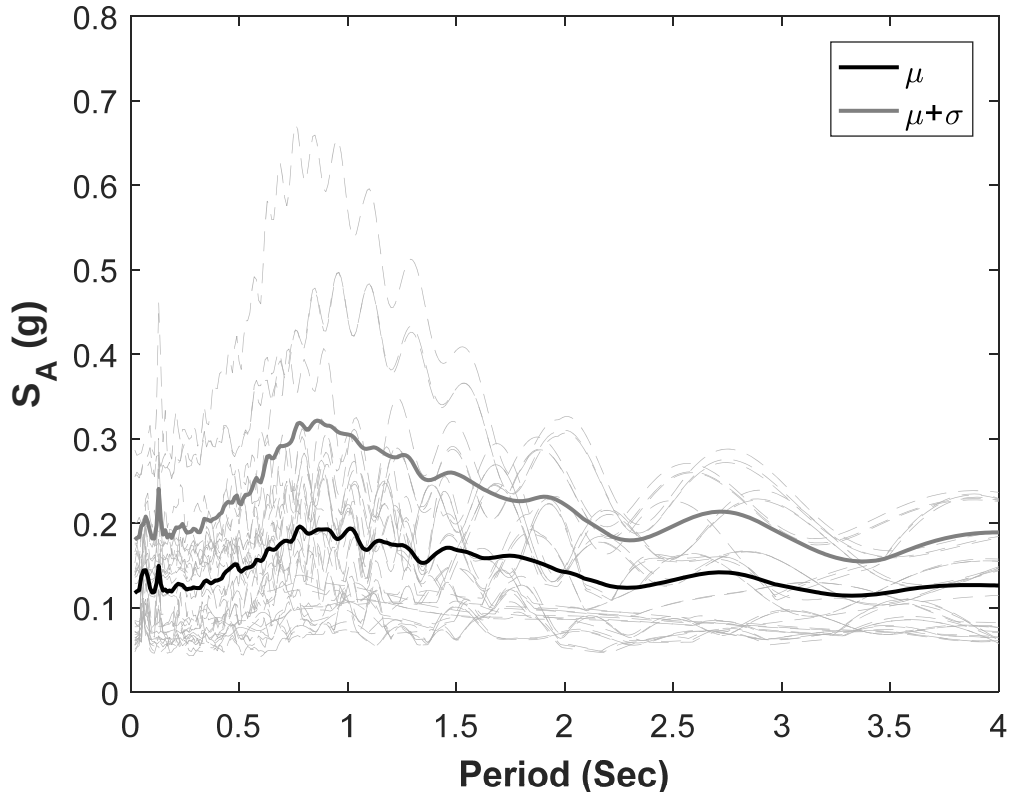


Figure 5.5.2-3 Response Spectra Mean and Mean Plus One Standard Deviation for Horizontal Accelerations, 100 T

5.6 Design Spectra Development and Guideline Recommendations

The spectra developed in the previous section based on individual load cases and motion cases. For design purposes, the spectra need to be generalized based on the payload relationship. This section presents the methodology for developing the Vertical and Horizontal Design Spectra.

5.6.1 Vertical Design Spectra

The vertical design assumptions are that of rigid falsework and flexible bridge. When the SPMT and the cargo are subjected to vertical acceleration, the platform transmits the vibration through rigid diaphragms and connections to the bridge. Therefore, assuming a rigid false work during the vertical bouncing is realistic and the PPA for vertical accelerations (PPA_V) can be used for falsework design and the spectral acceleration (S_{Amax}) is used to determine the dead load amplification on the bridge structure.

The PPA_V for each load case are plotted in Figure 5.6.1-1 and include a best fit exponential curve suitable for design. The proposed design equation for the PPA_V based on the percent of SPMT capacity (C) is as follows:

$$PPA_V = 0.7533e^{-0.03C} \qquad \text{(EQ. 5.6.1-1)}$$

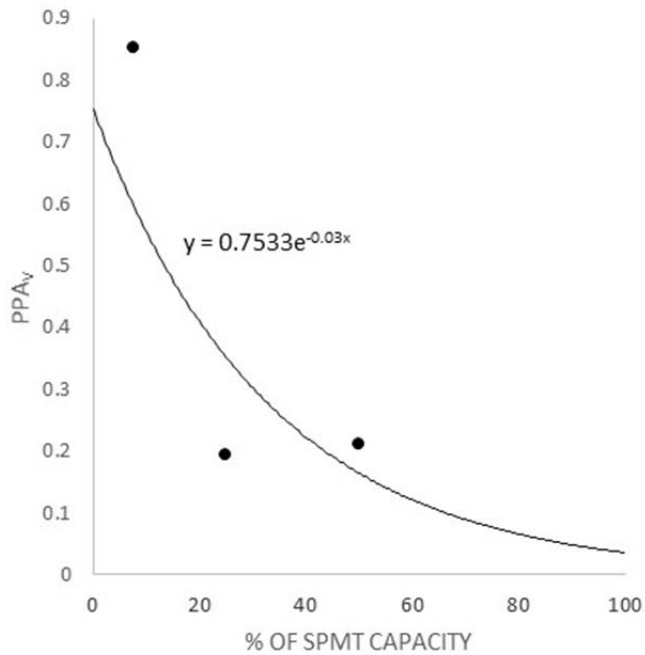


Figure 5.6.1-1 PPAV Versus % SPMT Capacity

To generate the design spectrum for vertical acceleration, the design spectral acceleration (S_{Amax}) is calculated with following equation:

$$S_{Amax} = \begin{cases} 3 \times PPA_V & 0 \leq T_n \leq T_1 \\ 3\left(\frac{T_1}{T_n}\right) PPA_V & T_n > T_1 \end{cases} \qquad \text{(Eq. 5.6.1 -2)}$$

Where T_1 is equal to 0.5 second and T_n is the nominal period of the to be analyzed structure. The shape of S_{Amax} is provided in Figure 5.6.1-2. The spectral acceleration is only a function of PPAv and the period of the structure.

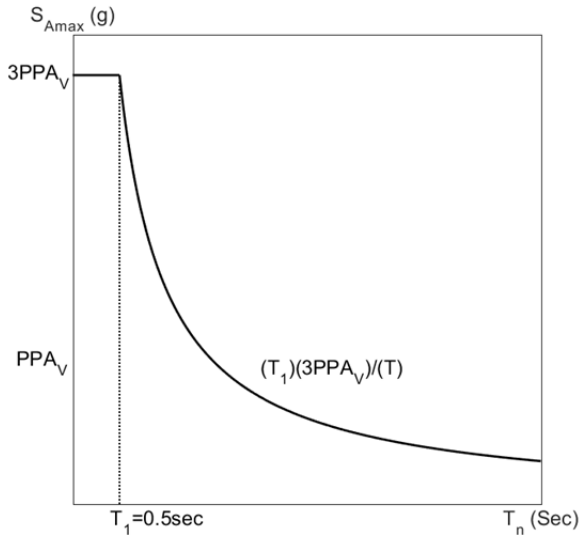


Figure 5.6.1-2 Proposed Vertical Acceleration Design Spectrum

For comparison purposes Figure 5.6.1-3 through Figure 5.6.1-5 present the Vertical Design Spectrum overlaid on the computed time history spectra, the $\mu + \sigma$ composite spectrum developed in the previous sections, for each load case. Notice that the ramp up from zero period to $3PPA_V$ is neglected for the design spectrum for simplicity and to discourage designers from underestimating the period of the structure to obtain lower loading.

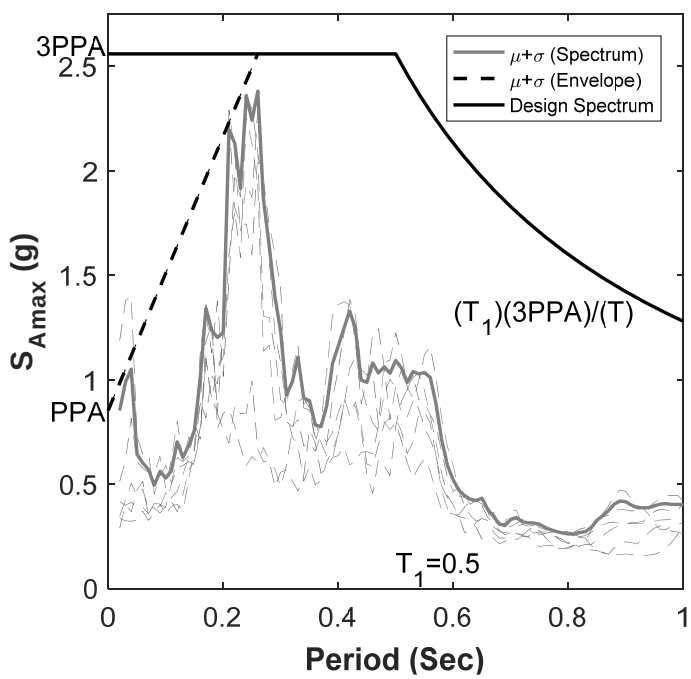


Figure 5.6.1-3 Design Spectrum for Vertical Acceleration on Light Load Case Spectra

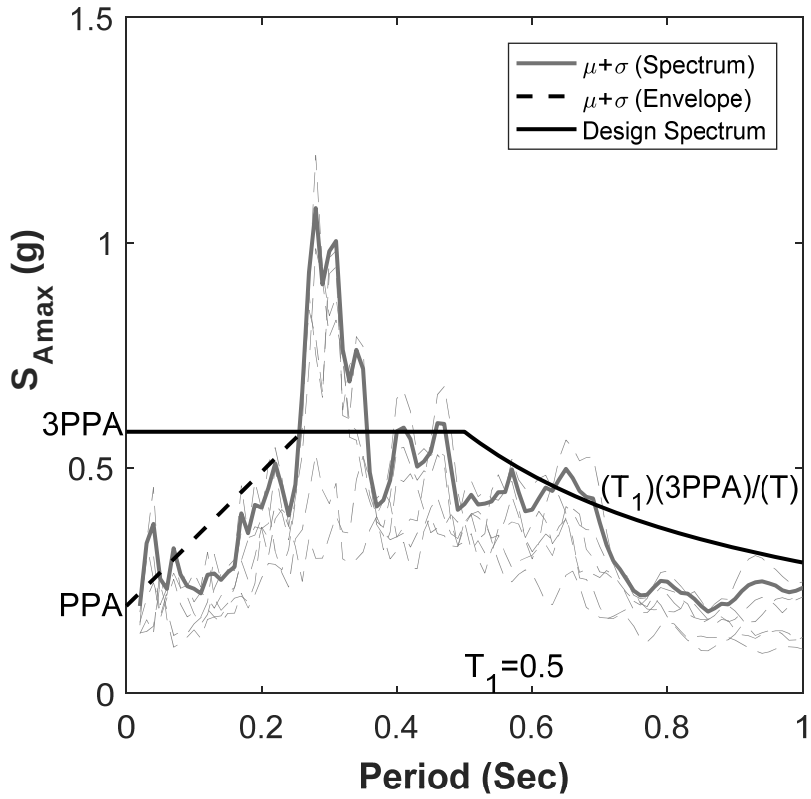


Figure 5.6.1-4 Design Spectrum for Vertical Acceleration on Medium Load Case Spectra

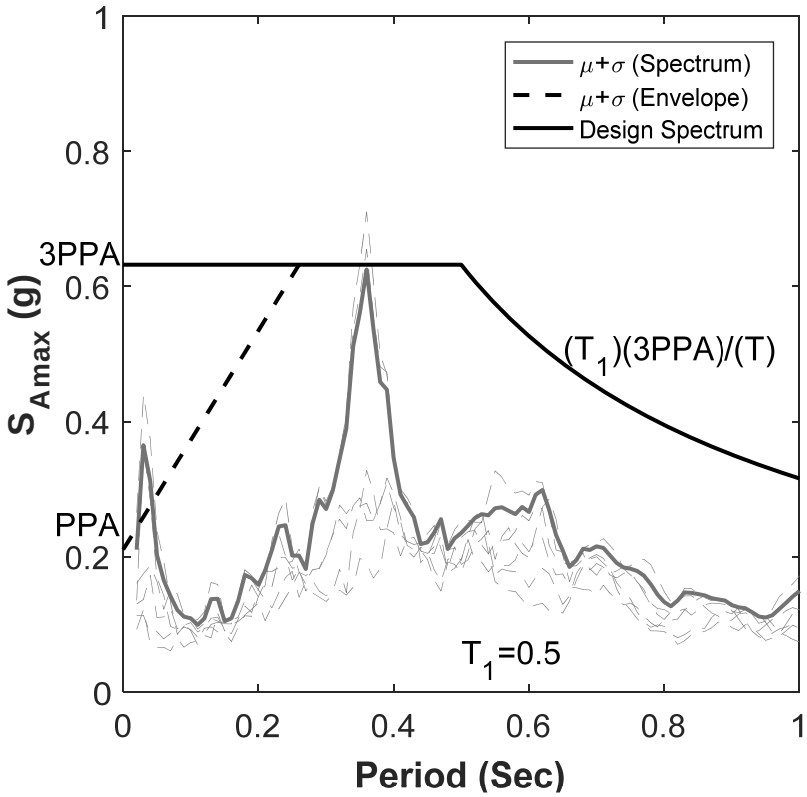


Figure 5.6.1-5 Design Spectrum for Vertical Acceleration on Heavy Load Case Spectra

As discussed previously, the value of PPA_V is dependent on the velocity of the SPMT. Because the vertical accelerations are caused by the roughness of the driven surface, this is a reasonable assumption. If the driving surface was assumed to be a sine wave displacement excitation or any other periodic excitation (see Figure 5.6.1-6.), then the displacement due to the sine wave can be expressed in following equation:

$$u(t) = D\sin\left(\frac{2\pi}{T}t\right) \qquad \text{(Eq. 5.6.1-3)}$$

Where D is the amplitude of the roadway roughness, T is the period of the sine wave function and t is time. Then the velocity *V,* and acceleration *A*, are first and second derivative of Eq. (5.6.1-3):

$$V(t) = D\left(\frac{2\pi}{T}\right)\cos\left(\frac{2\pi}{T}t\right) \qquad \text{(Eq. 5.6.1-4)}$$

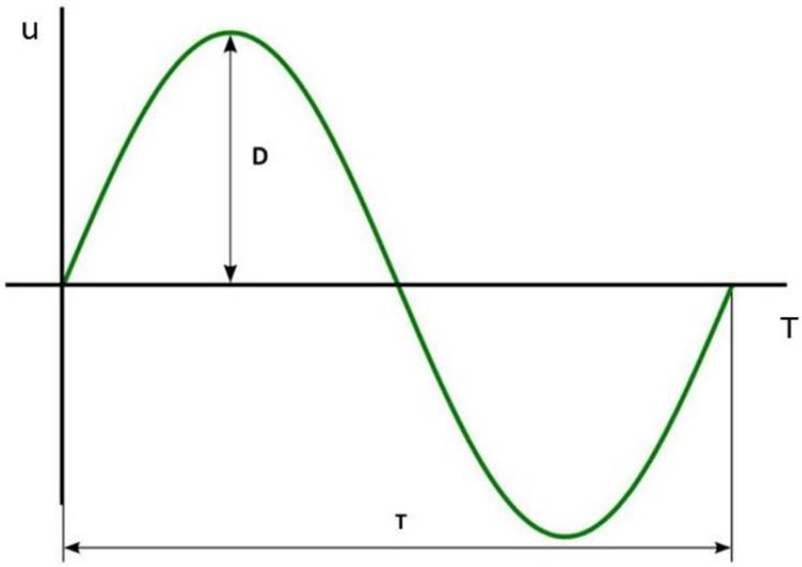


Figure 5.6.1-6 The Sine Wave Excitation

Therefore, a change in velocity would affect the acceleration by the power of 2. In other words, if the vertical acceleration measured at velocity of V is equal to A, then the vertical acceleration at V/2 would be equal to A/4.

During the Long Run Load Case, the SPMT speed was estimated at 4 mph, or a fast walk, however, functionally, this is unlikely in practice where it is likely limited to 2.5 mph. Using this information, the design accelerations can be modified in the proposed specification. This is handled using by using a load reduction factor of $(2.5 \text{ mph}/4 \text{ mph})^2 = 0.4$ to account for service limit state and a worst case 1.0 for the ultimate limit state.

5.6.2 Horizontal Design Spectra

The horizontal design assumption is that of a rigid bridge/cargo (i.e., rigid diaphragm action) and flexible falsework. When the SPMT is subjected to the horizontal deceleration, the platform transmits the impulse through the falsework and into the bridge which will be minimally affected by the horizontal decelerations. Therefore the critical design case for horizontal accelerations is the falsework and the S_{Amax} for the horizontal direction, outlined below should be used.

The PPA for horizontal accelerations (PPA_H) for each load case are presented in Figure 5.6.2-1 and include the best fit exponential approximation proposed for design. The proposed design equation for the PPA_H, based on the percent of SPMT capacity is as follows:

$$PPA_H = 0.3615e^{-0.014C} \qquad \text{(Eq. 5.6.2-1)}$$

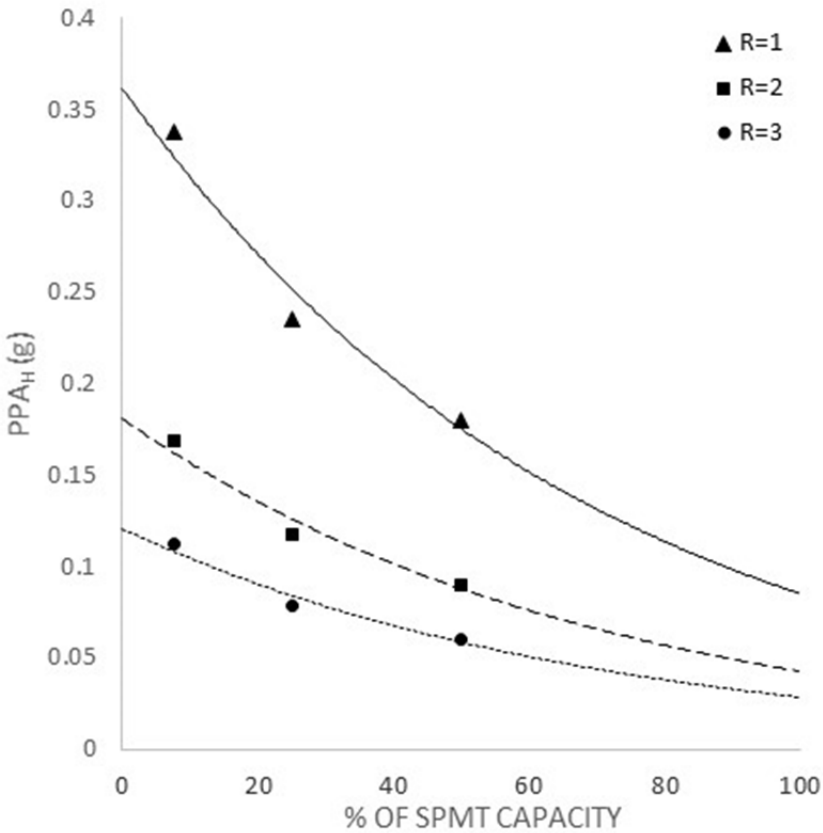


Figure 5.6.2-1 PPAH Versus % SPMT Capacity

To generate the design spectrum for horizontal acceleration, presented in Figure 5.6.2-2, PPA_H is obtained from Eq. (5.6.2-1) and Eq. (5.6.2-2) is used:

$$S_{Amax} = \begin{cases} 2 \times PPA_H & 0 \le T_n \le T_1 \\ 2\left(\frac{T_1}{T_n}\right) PPA_H & T_n > T_1 \end{cases} \qquad \text{(Eq. 5.6.2-2)}$$

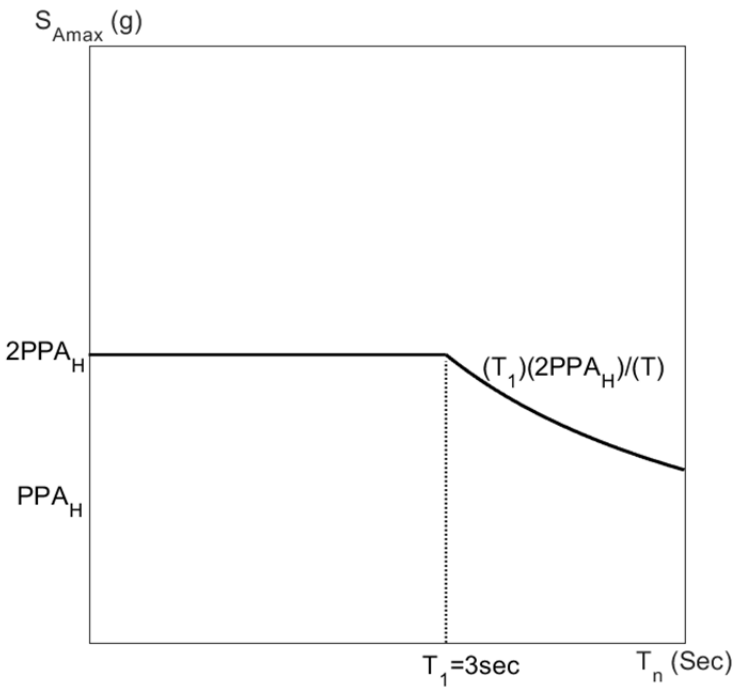


Figure 5.6.2-2 Proposed Design Spectrum for Horizontal Acceleration

For comparison purposes, Figure 5.6.2-3 through Figure 5.6.2-5 present the horizontal design spectrum overlaid on the computed time history spectra and the $\mu + \sigma$ composite spectrum developed in the previous sections, for each load case. Notice that the ramp up from zero period to $2PPA_V$ is neglected for the design spectrum for simplicity and to discourage designers from underestimating the period of the structure to obtain lower loading.

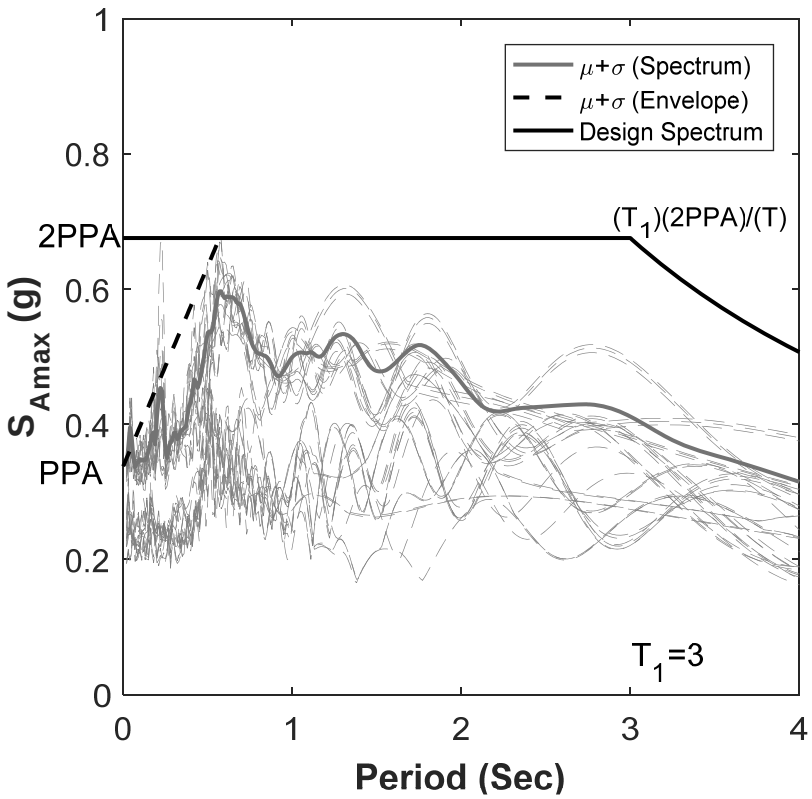


Figure 5.6.2-3 Design Spectrum for Horizontal Acceleration on Light Load Case Spectra

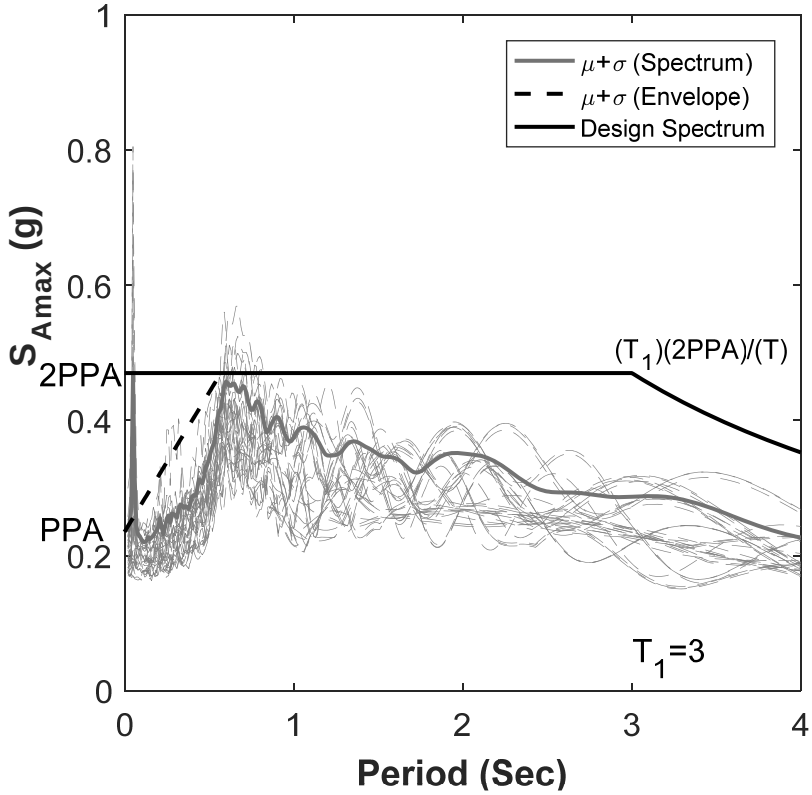


Figure 5.6.2-4: Design spectrum for horizontal acceleration on medium load case spectra

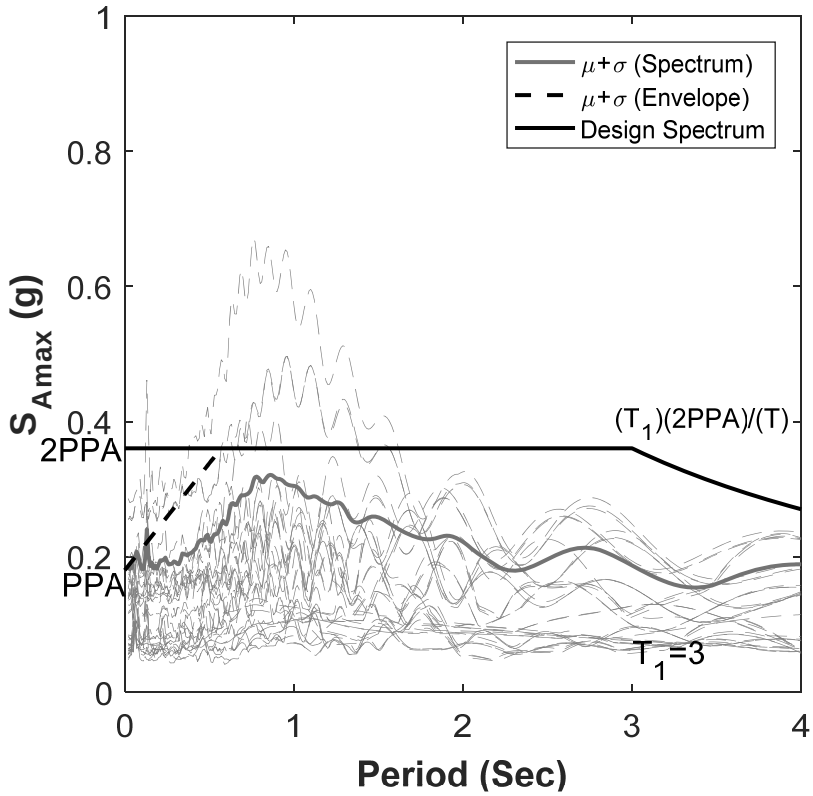


Figure 5.6.2-5 Design Spectrum for Horizontal Acceleration on Heavy Load Case Spectra

Justification for reducing the vertical accelerations for the service limit states was presented in the previous section. However, for the horizontal case, a similar justification cannot be made. Furthermore, since the horizontal accelerations apply mostly to the falsework, which is a temporary structure, some level of damage is considered acceptable. For this reason the falsework designer shall be allowed to use the response modification ratio (R) to modify the Eq. 5.6.2-2. It is recommended that if the bridge supports have limited ductility (rigid) use R = 2 and if the bridge supports have medium ductility (semi rigid) use R = 2.5. The basis of these recommendations is the *AASHTO Guide Design Specifications for Bridge Temporary Works,* which calls for an R Factor of 2.5 for temporary bracing that is only designed for strength and not ductility. The conservative value of 2.0 was chosen to reflect the variations in bracing systems in use when compared to typical temporary bracing systems. Demonstration of how to use the R values is presented in Figure 5.6.2-1.

5.7 Conclusions

In the preceding sections, the dynamic response of lightly and heavily loaded SPMTs was investigated during several common maneuvers. It was found that traversing a long stretch of uneven ground was critical for the vertical direction (the Long Run Load Case) and rapid decelerations were critical for the horizontal direction (the Start and Stop Load Case). Transverse accelerations were found to not be critical for the horizontal direction. Increasing the payload of the SPMT was found to decrease measured accelerations significantly. Acceleration data was recorded from the platform in various locations during

these maneuvers. The data collected was used to develop design response spectra, treating the problem similarly to an earthquake design scenarios.

The following conclusions can be made based on the testing and analysis of SPMT dynamic loads:

- Vertical accelerations were controlled by the Long Run Load Case.
- Vertical accelerations are related to SPMT transport speed.
- Vertical accelerations affect the falsework and the cargo differently: The cargo is considered flexible for design and the falsework can be considered rigid. The Vertical Design Spectrum is recommended for vertical design of the flexible cargo and applying the PPAV to the falsework is considered acceptable.
- Horizontal accelerations were controlled by the longitudinal direction of the Start and Stop Load Case.
- Horizontal accelerations are not affected by the speed of the SPMT, but are affected by the ability of the operator to perform a braking operation.
- Horizontal accelerations affect the falsework and the cargo differently: The cargo is considered rigid for design and the falsework is considered flexible. The horizontal design spectrum is recommended for horizontal design of the flexible falsework and horizontal forces acting on the rigid bridge are considered negligible.
- Transverse accelerations were considerably lower than the longitudinal accelerations, therefore longitudinal accelerations are recommended for both horizontal directions when designing.
- Significant reductions in PPA were measured as payload increased. Contractors are recommended to use large loads, with respect to SMPT capacity, to reduce design forces.
- Design spectra were developed for the horizontal and vertical directions, which are based only on an estimation of PPA for the SPMT loading.
- Justifications for reducing PPA and the design spectra were presented for both directions:
    - Vertical accelerations can be reduced by limiting the speed of the SPMT during transport
    - Horizontal accelerations can be reduced because some level of damage is likely acceptable to the falsework and response modification factors may be applied to the developed spectra, similar manner currently used for AASHTO LRFD earthquake design.

5.8 Data Plots

This section contains plots of the various data sets that were used for the development of this report.

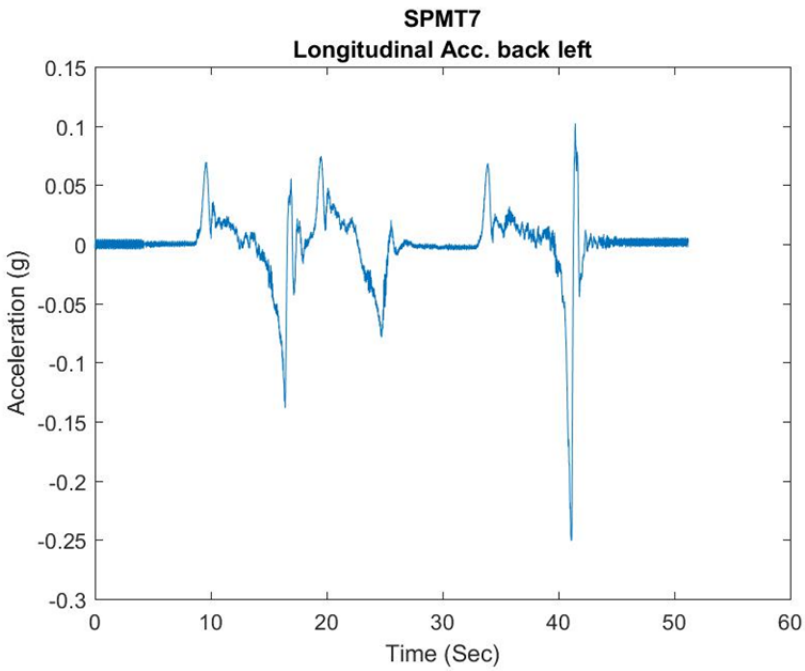


Figure 5.8-1 The Time History for Maximum Longitudinal Acceleration for Cluster 1 in Heavy Load Case

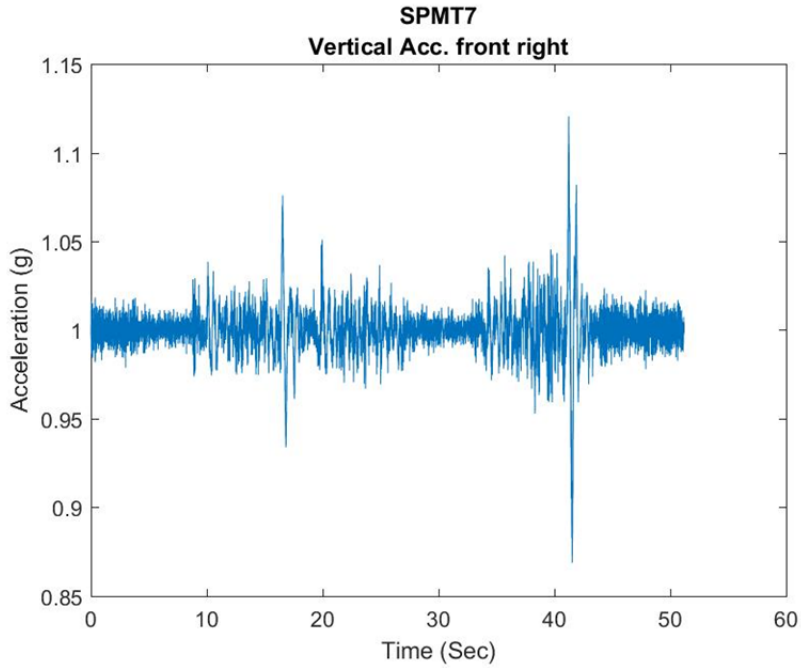


Figure 5.8-2 The Time History for Maximum Vertical Acceleration for Cluster 1 in Heavy Load Case

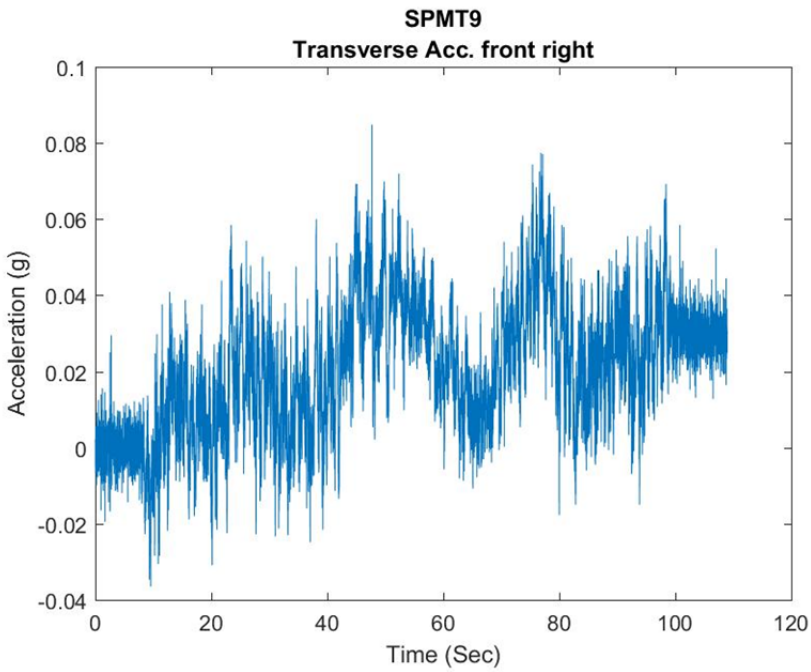


Figure 5.8-3 The Time History for Maximum Transverse Acceleration for Cluster 1 in Heavy Load Case

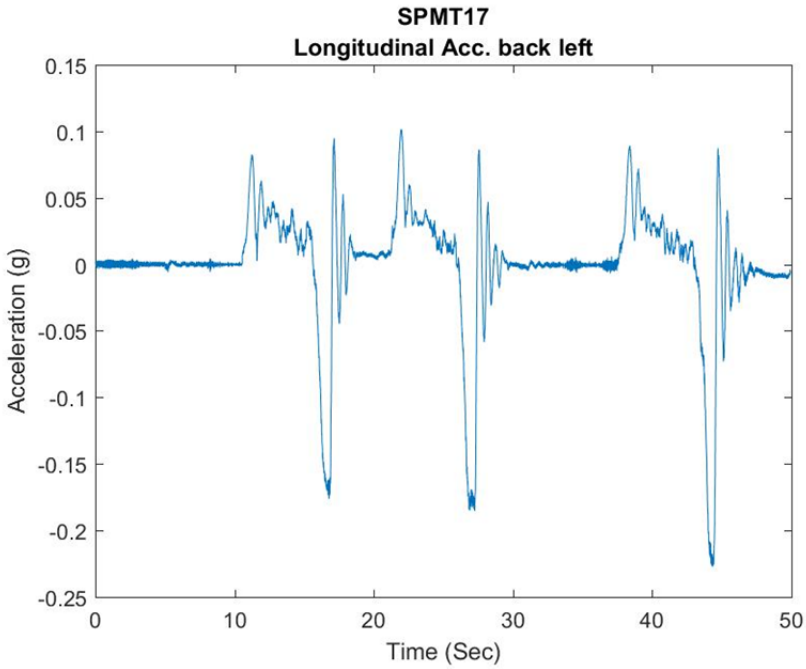


Figure 5.8-4 The Time History for Maximum Longitudinal Acceleration for Cluster 1 in Medium Load Case

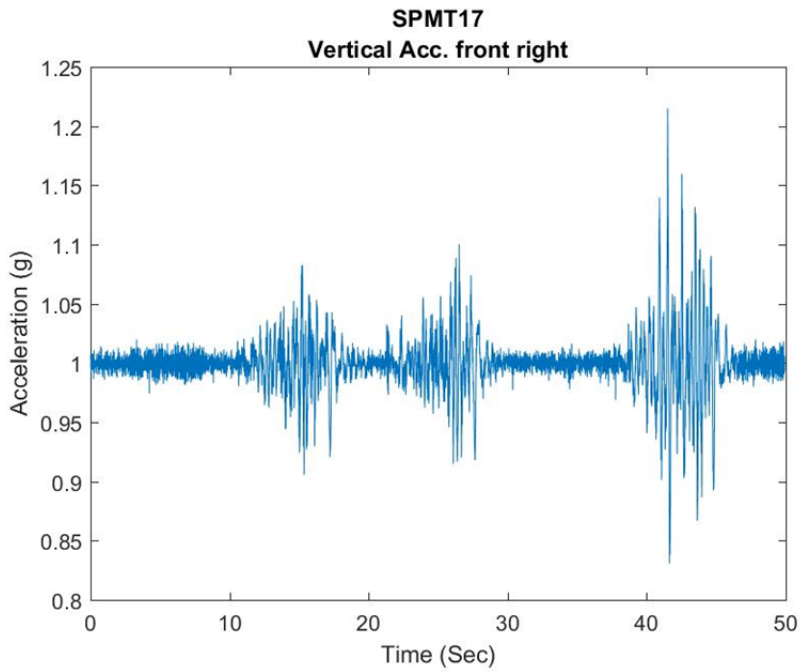


Figure 5.8-5 The Time History for Maximum Vertical Acceleration for Cluster 1 in Medium Load Case

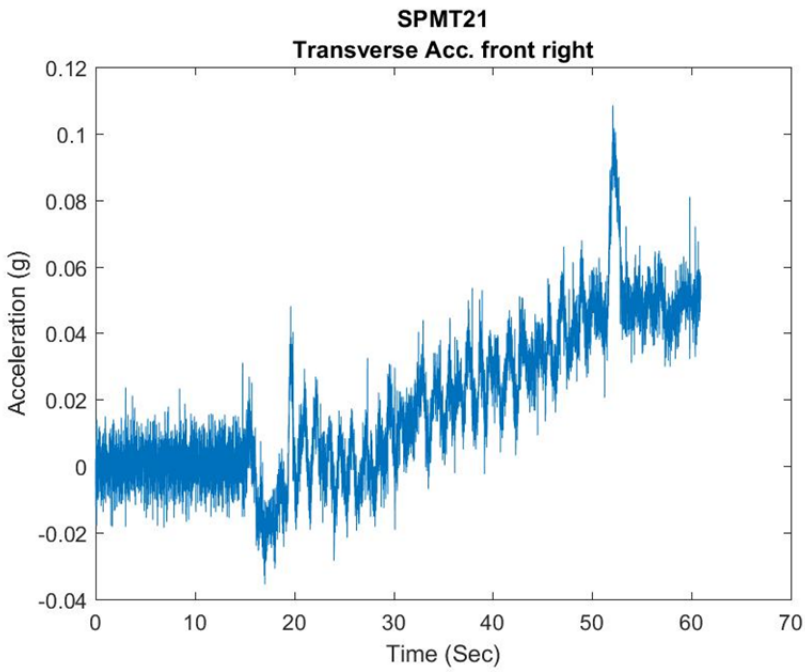


Figure 5.8-6 The Time History for Maximum Transverse Acceleration for Cluster 1 in Medium Load Case

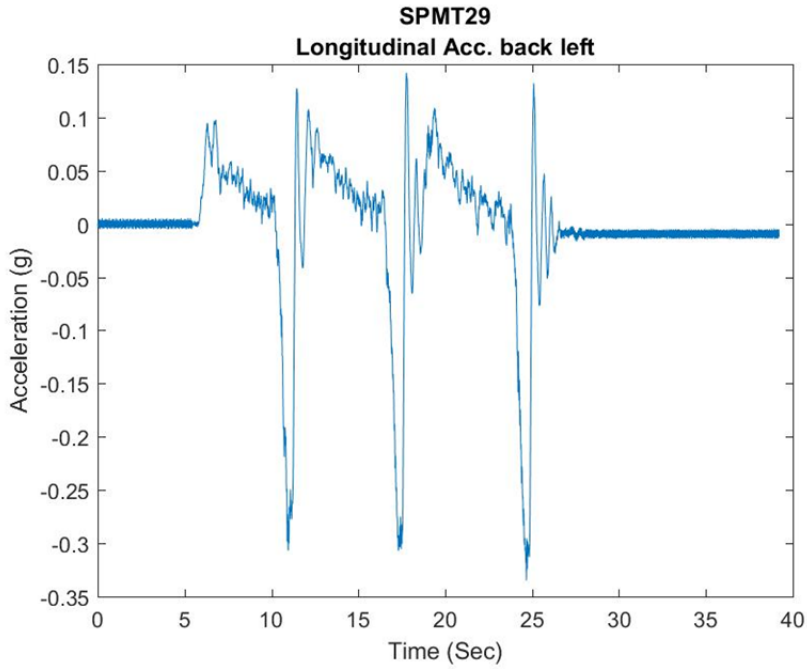


Figure 5.8-7 The Time History for Maximum Longitudinal Acceleration for Cluster 1 in Light Load Case

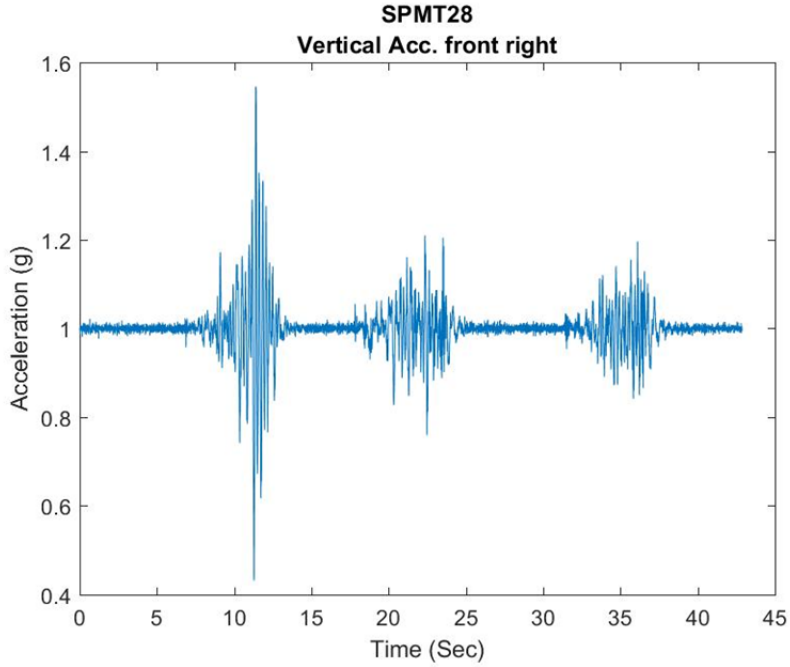


Figure 5.8-8 The Time History for Maximum Vertical Acceleration for Cluster 1 in Light Load Case

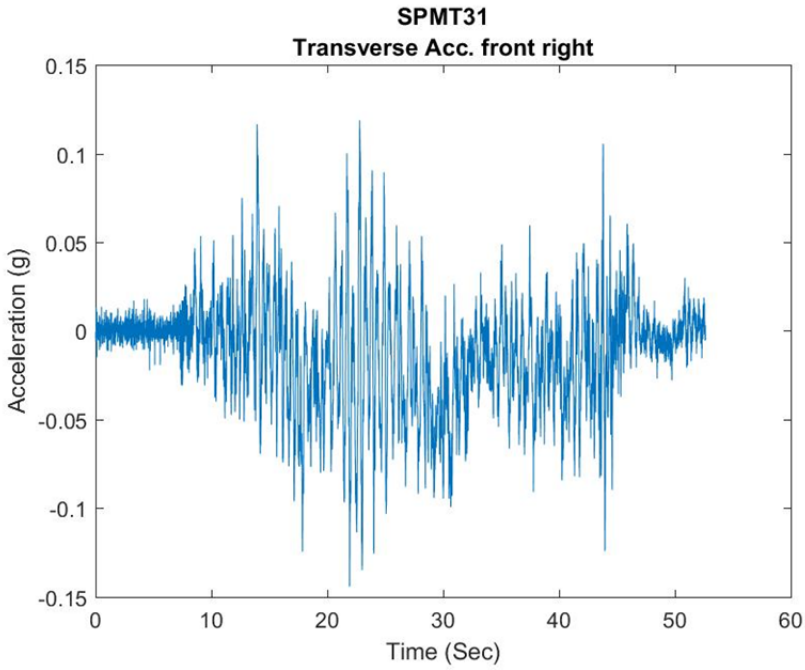


Figure 5.8-9 The Time History for Maximum Transverse Acceleration for Cluster 1 in Light Load Case

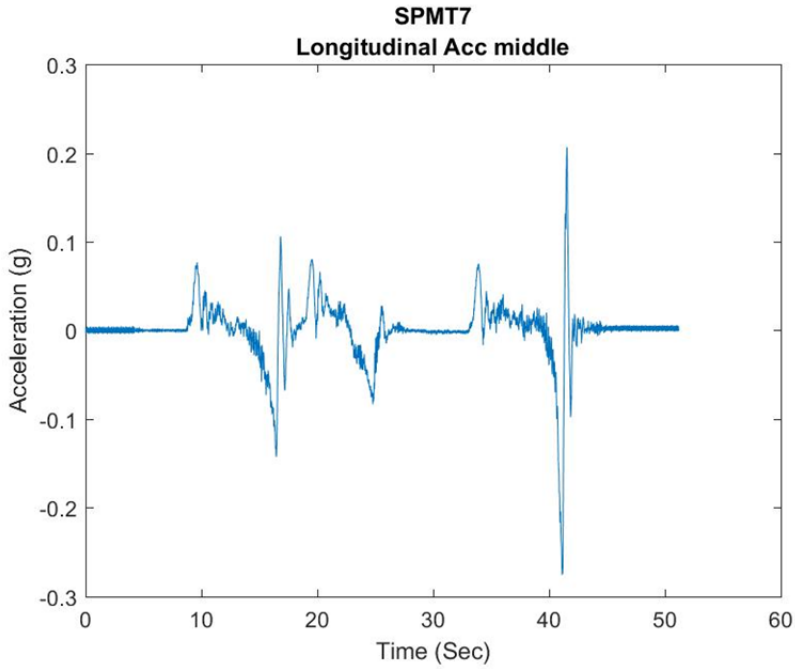


Figure 5.8-10 The Time History for Maximum Longitudinal Acceleration for Cluster 2 in Heavy Load Case

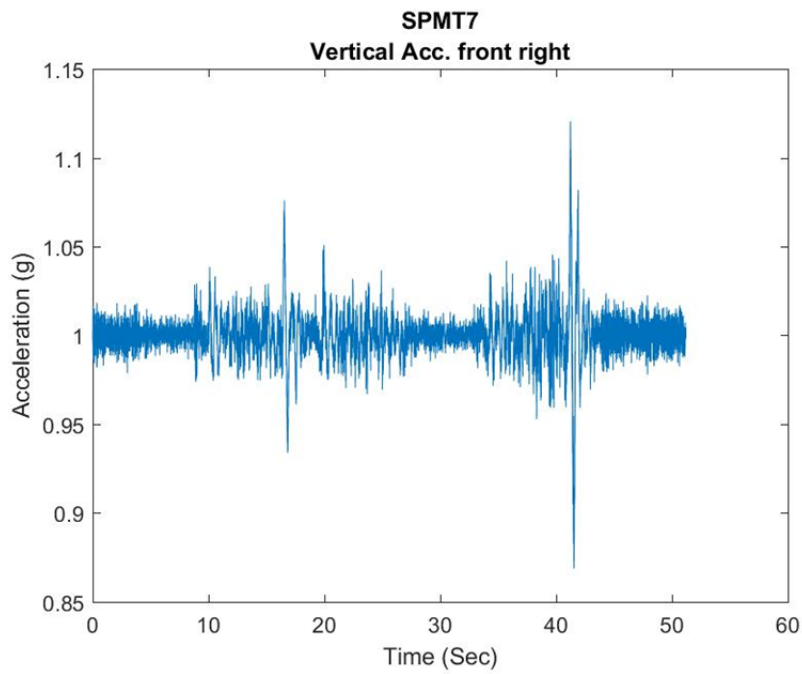


Figure 5.8-11 The Time History for Maximum Vertical Acceleration for Cluster 2 in Heavy Load Case

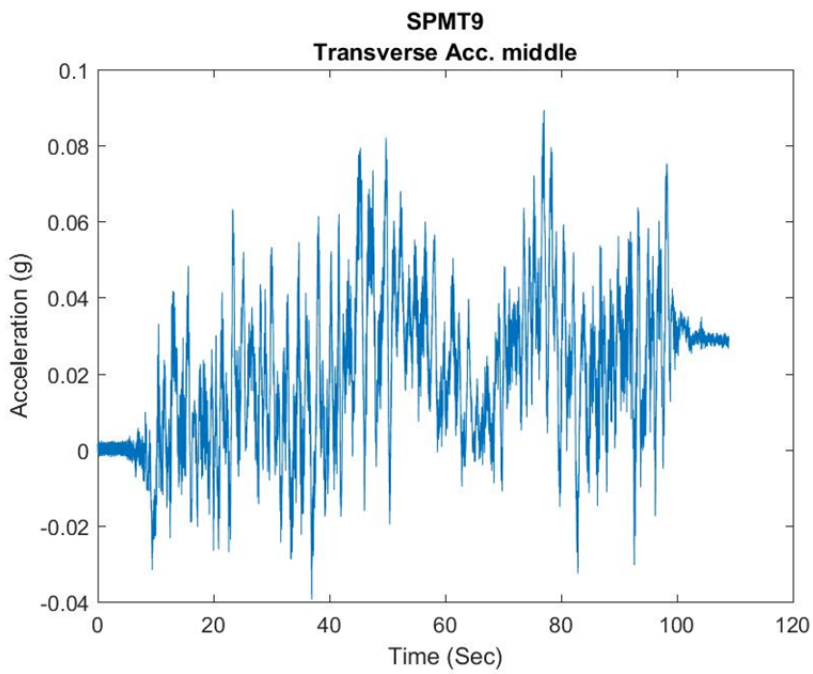


Figure 5.8-12 The Time History for Maximum Transverse Acceleration for Cluster 2 in Heavy Load Case

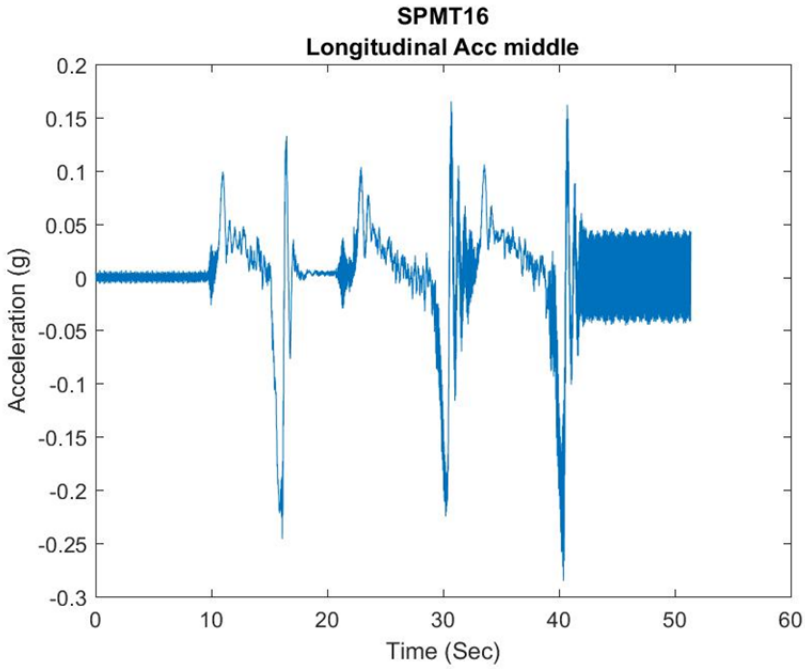


Figure 5.8-13 The Time History for Maximum Longitudinal Acceleration for Cluster 2 in Medium Load Case

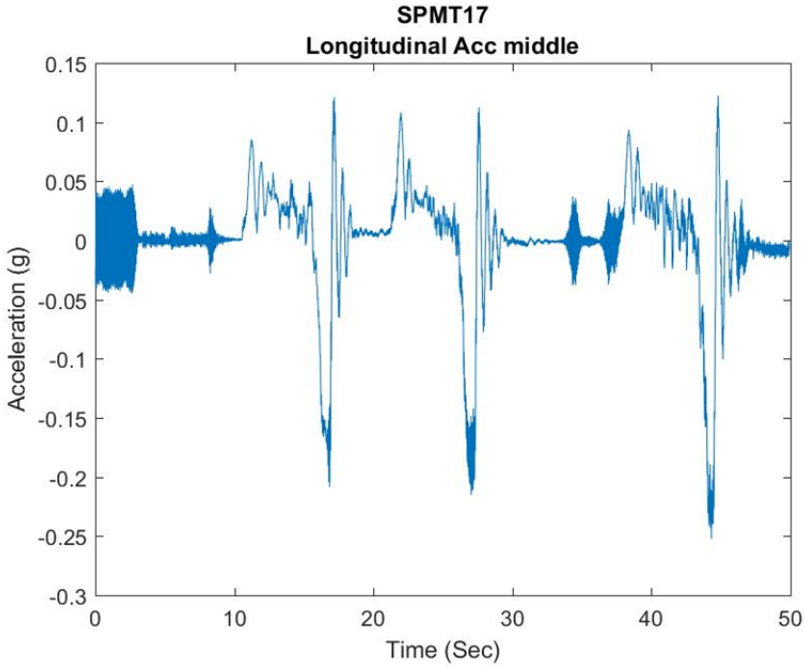


Figure 5.8-14 The Time History for Maximum Vertical Acceleration for Cluster 2 in Medium Load Case

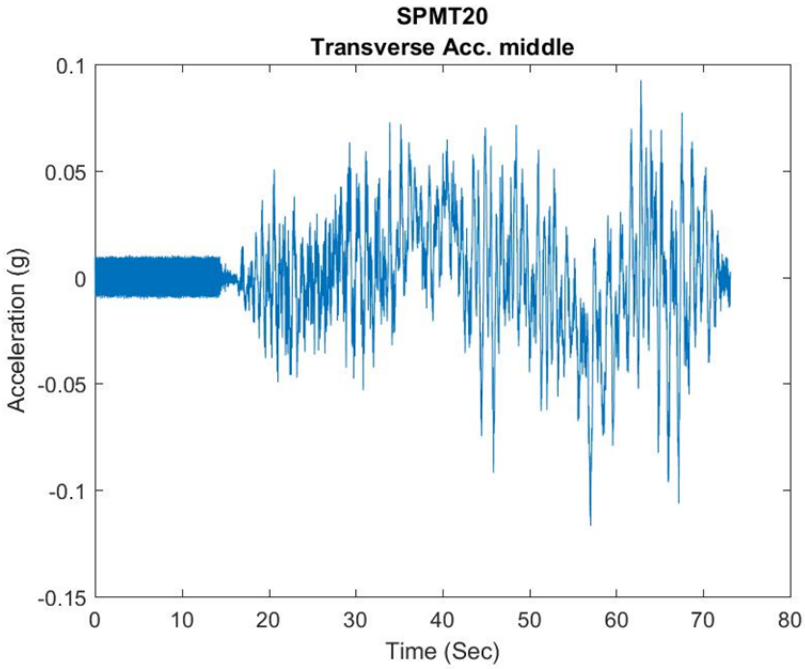


Figure 5.8-15 The Time History for Maximum Transverse Acceleration for Cluster 2 in Medium Load Case

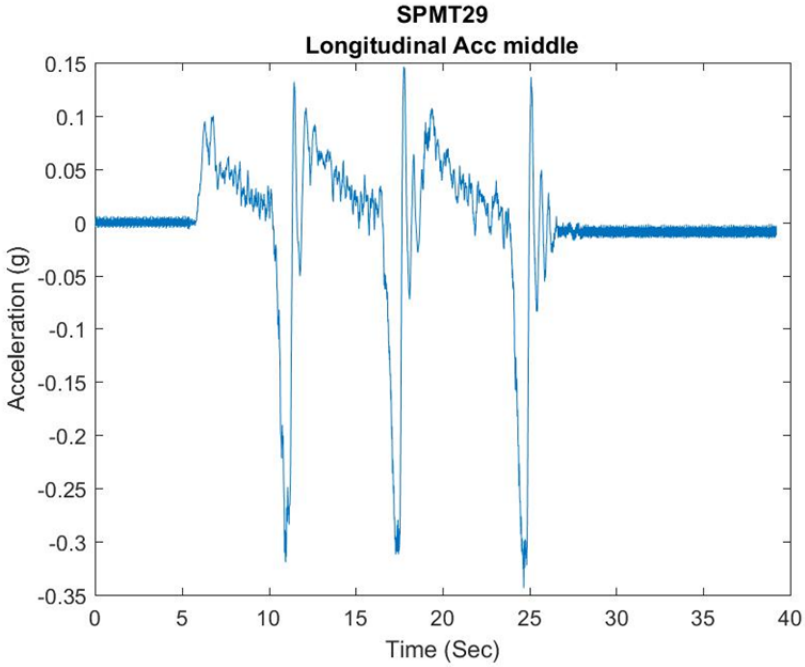


Figure 5.8-16 The Time History for Maximum Longitudinal Acceleration for Cluster 2 in Light Load Case

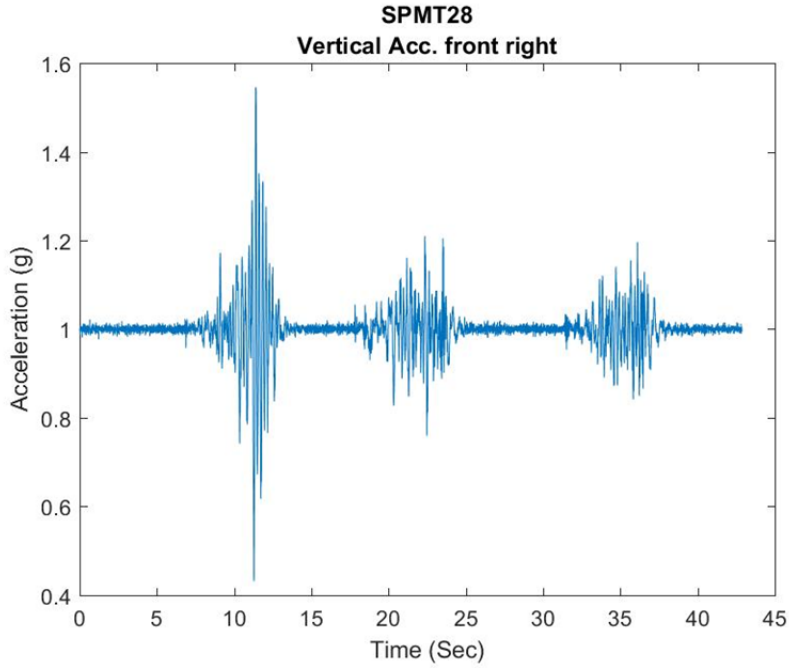


Figure 5.8-17 The Time History for Maximum Vertical Acceleration for Cluster 2 in Light Load Case

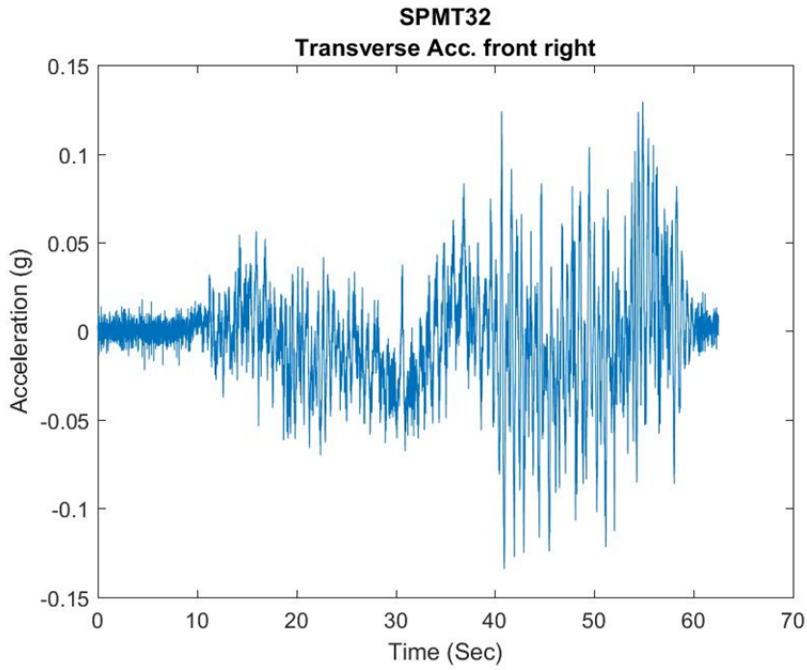


Figure 5.8-18 The Time History for Maximum Transverse Acceleration for Cluster 2 in Light Load Case

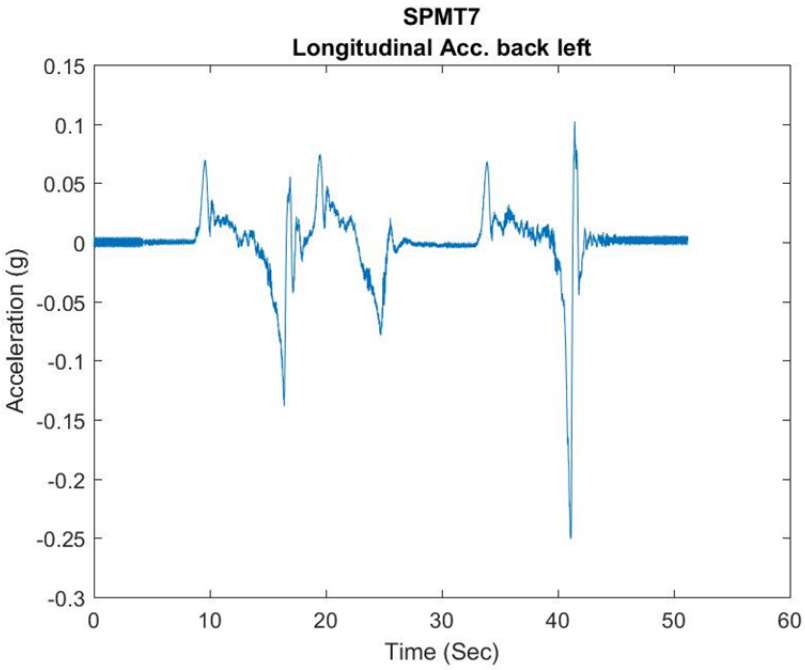


Figure 5.8-19 The Time History for Maximum Longitudinal Acceleration for Cluster 3 in Heavy Load Case

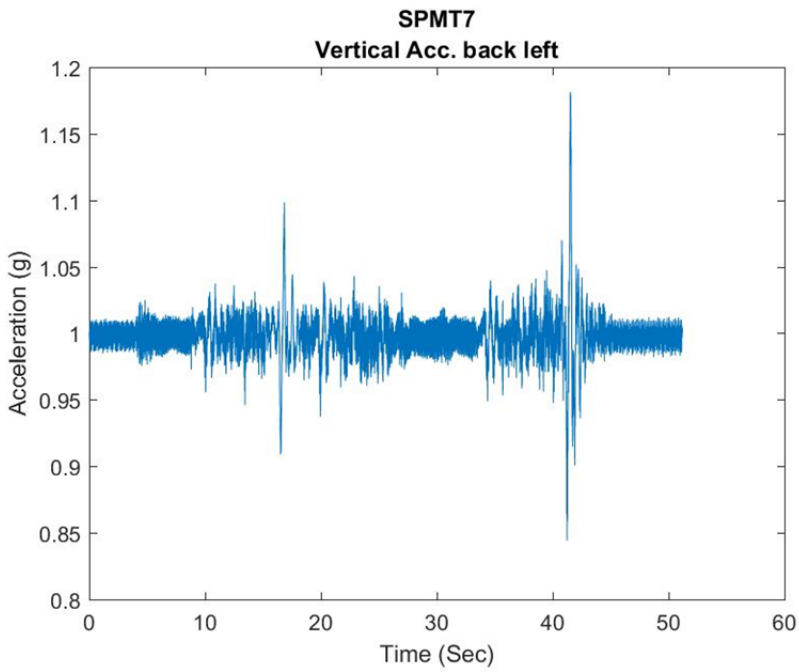


Figure 5.8-20 The Time History for Maximum Vertical Acceleration for Cluster 3 in Heavy Load Case

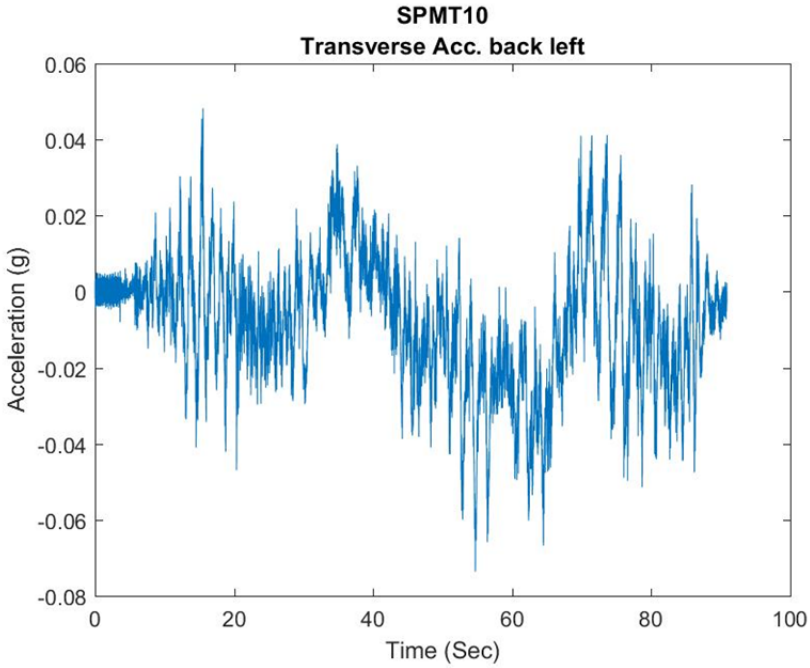


Figure 5.8-21 The Time History for Maximum Transverse Acceleration for Cluster 3 in Heavy Load Case

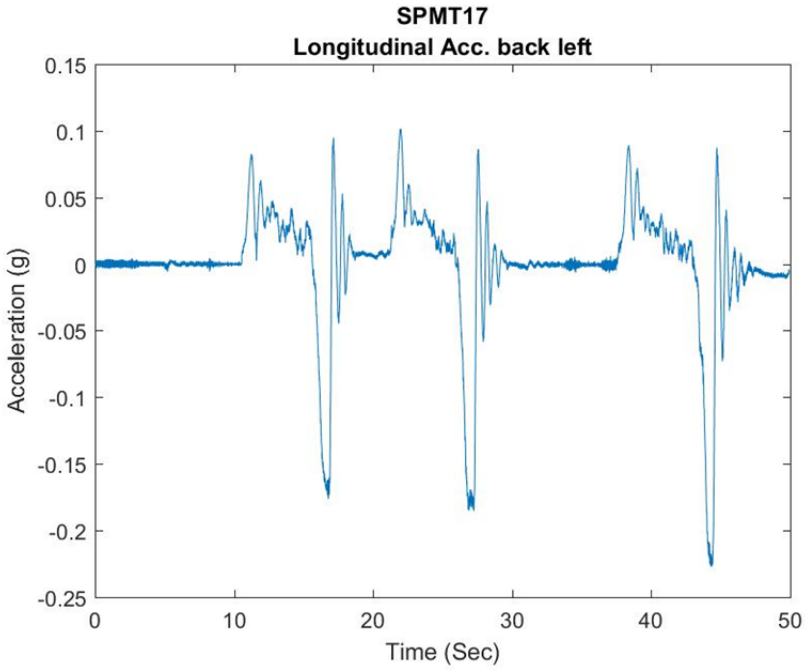


Figure 5.8-22 The Time History for Maximum Longitudinal Acceleration for Cluster 3 in Medium Load Case

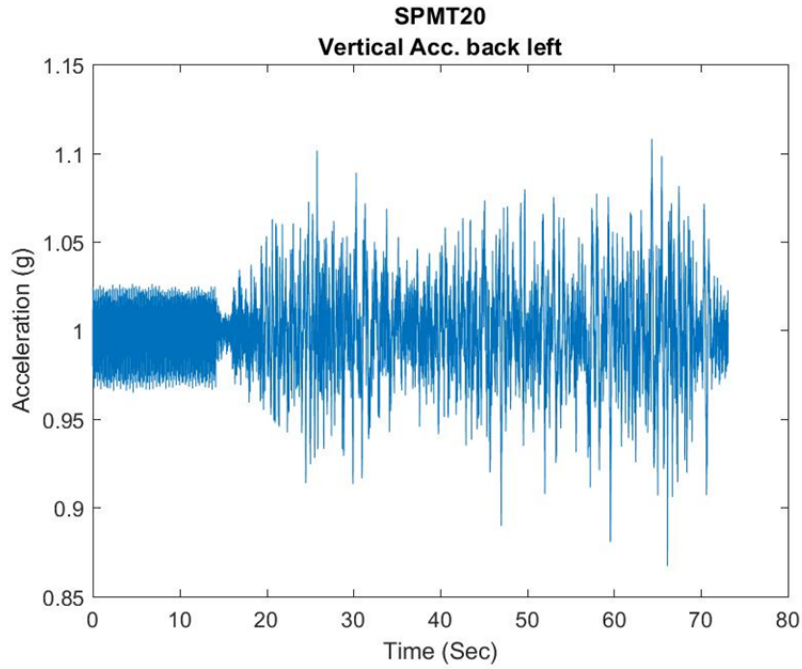


Figure 5.8-23 The Time History for Maximum Vertical Acceleration for Cluster 3 in Medium Load Case

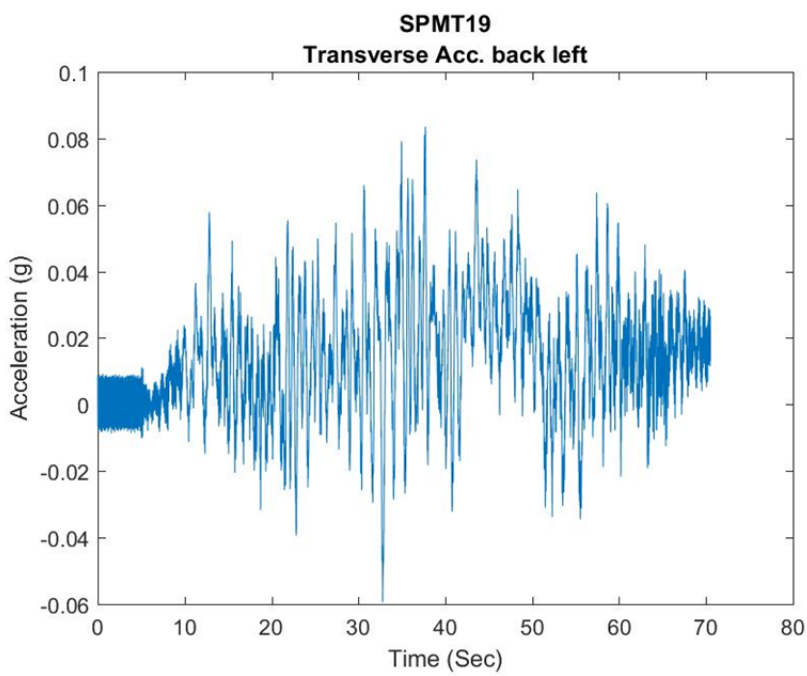


Figure 5.8-24 The Time History for Maximum Transverse Acceleration for Cluster 3 in Medium Load Case

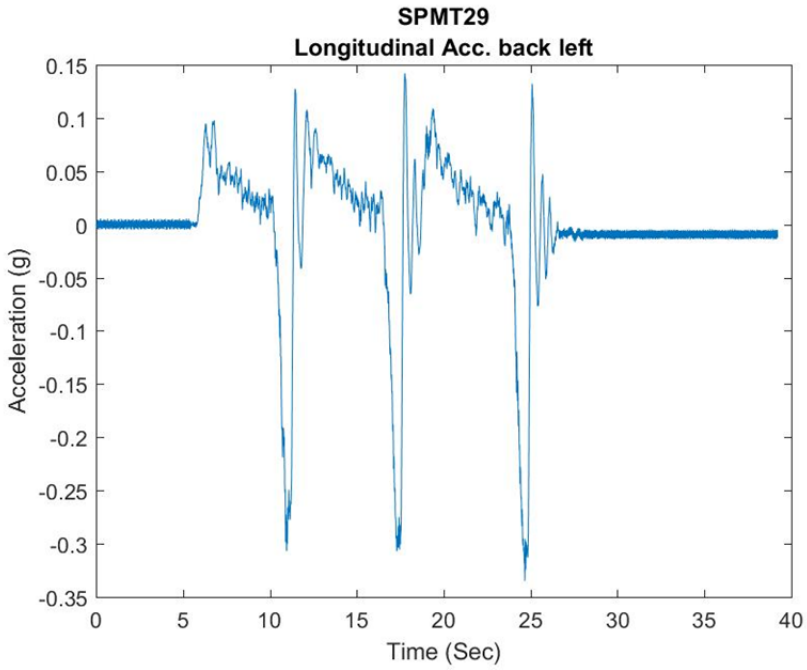


Figure 5.8-25 The Time History for Maximum Longitudinal Acceleration for Cluster 3 in Light Load Case

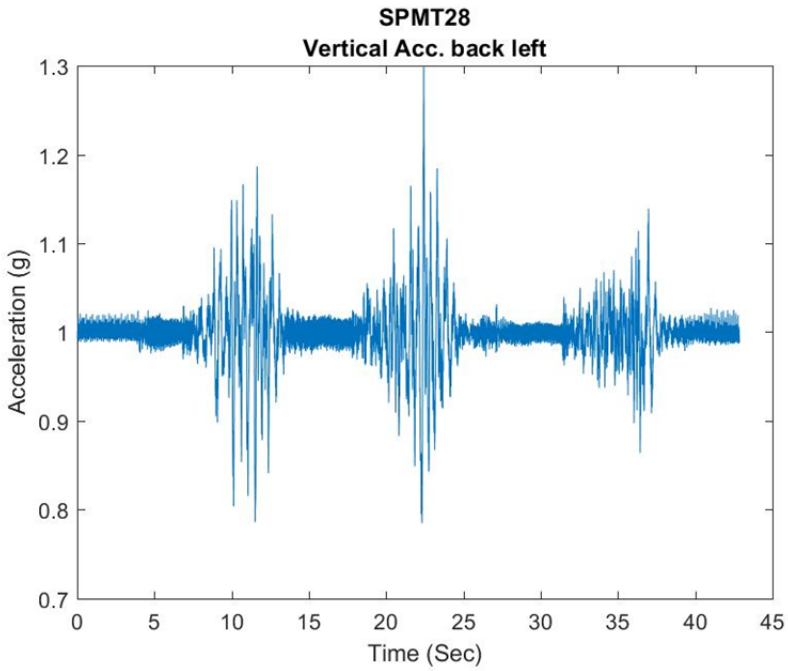


Figure 5.8-26 The Time History for Maximum Vertical Acceleration for Cluster 3 in Light Load Case

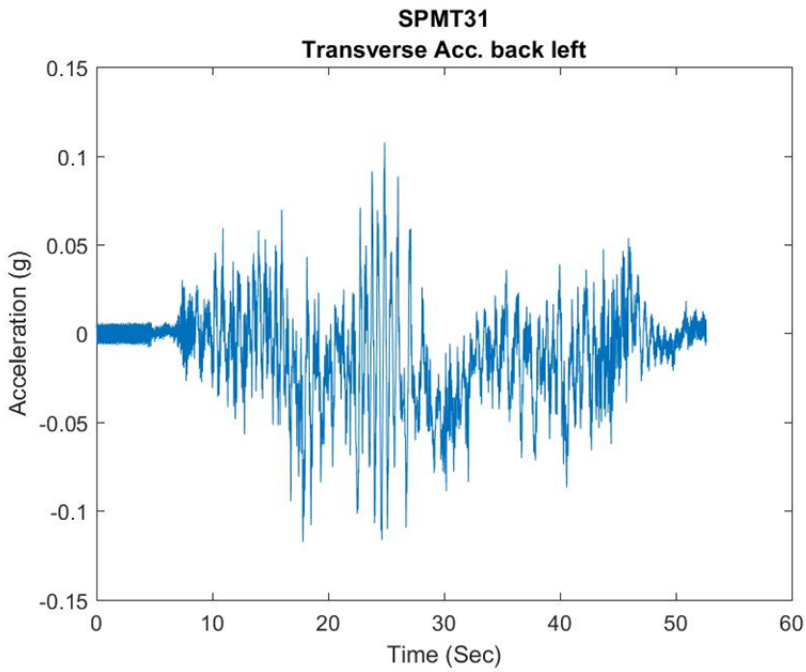


Figure 5.8-27 The Time History for Maximum Transverse Acceleration for Cluster 3 in Light Load Case

CHAPTER 6

Investigation of Dynamics for Lateral Slide Bridge Systems

6.1 Introduction

Many states in the U.S. have successfully replaced existing bridges using SIBC, where the new bridge is built adjacent to the existing bridge on temporary falsework and slid into place upon removal of the existing bridge. A track system is set up from the falsework to the permanent abutments which is used to slide the bridge structure into place. It is common to use PTFE (polytetrafluoroethylene) on one surface and stainless steel on the other surface to minimize friction while sliding. PTFE is commonly referred to as Teflon® which is DuPont's brand name for PTFE.

Significant research has been done in the past by others for PTFE-stainless steel bridge bearings to obtain coefficients of friction (CoF). The majority of research has focused on bearings designed to permanently remain under the bridge structure. With SIBC, the bearings slide the bridge into place and are subsequently removed. Many of the parameters that influence the CoF will be similar and can be compared to results obtained by researchers studying permanent bearings. However, the range of some of the parameters studied by others may not apply to bearings used for SIBC. For example, sliding velocity is a parameter that affects the CoF and will be tested, but it does not need to be tested at dynamic speeds, as others have done, to simulate live traffic or seismic activity. There are 14 parameters that previous researchers identified that may influence the CoF (35):

- Type of PTFE
- Dimpling and lubrication
- Roughness of the mating surface
- Contact pressure
- Sliding velocity
- Length of the travel path
- Temperature
- Specimen size
- Attachment of the PTFE to the backing plate
- Eccentric loading
- Load and travel history
- Surface contamination
- Wear
- Creep

The roughness of the mating surface as well as several other parameters were studied by Taylor and Stanton (2010). The main purpose of their research was to determine the CoF for 3 separate stainless steel finishes with PTFE. Current *AASHTO LRFD Bridge Construction Specifications* require the use of stainless steel with a finish of better than or equal to 8 μin (rms). This is referred to as "No.8 mirror finish" which is the most reflective and smoothest stainless steel covered by ASTM standards. Because

No.8 mirror finish is much more expensive and less available than other finishes, 2 other finishes were investigated. No.2B stainless steel, which is only lightly polished and is much cheaper and more common than No.8 mirror was tested, as well as RR stainless steel which is not polished at all.

Conclusions were made that No.2B finish behaves similar to No.8 mirror finish but has a slightly higher CoF initially with slight surface changes as the slide path increases. The CoF after a long slide path of 1600 in. was lower than the CoF for the No 8 mirror finish. For the rough-rolled finish, initially the CoF was considerably higher than the No.8 and No.2B finishes, as anticipated. However, as the slide path increased, the friction decreased to where the CoF was lower than the No.8 finish, due to surface wear as the slide path increased. Taylor and Stanton (2010) concluded that the No.2B finish is a good alternative to No.8 mirror finish. These findings were compared to Campbell and Manning (1990), who found similar trends with unlubricated 'standard finish' stainless steel vs mirror finish.

Based on the available research, the authors plan to build upon and fill in the gaps. Some tests will be similar to tests done by previous researchers in order to compare results and have data to use as a reference. The authors propose that the test matrix, presented in Table 1, have the following 4 parameters in it that will be varied:

1. Roughness of mating surface
2. Lubrication and dimpling
3. Contact pressure
4. Sliding velocity

These are some of the parameters that the previous researchers have found to have significant influence on the CoF. The following will explain 4 initial parameters selected as an experimental starting point. Lessons from past research, on current AASHTO specifications and current construction practice are incorporated.

Roughness of Stainless Steel:

Past research shows that the roughness of mating surface is a major factor in the CoF and expense/availability of the materials. Taylor and Stanton (2010) demonstrated that it is likely more cost effective to not use highly polished stainless steel like AASHTO specifies.

Sliding Velocity:

Article 18.1.5.2.6 of the *AASHTO LRFD Bridge Design Specifications* states that the slide speed shall be 2.5 in./min. This speed was also used in the friction testing research by Taylor and Stanton (2010). For consistency, this base speed was used during testing, which allows for easy comparisons with historical data. The faster speed, 10 inches per minute, was also investigated because contractors may want to push a bridge at a faster rate.

Lubrication:

It is common knowledge that lubrication can significantly reduce the CoF. Dolce et al. (2005) found that stainless steel-PTFE interfaces frictional resistance can be reduced by a factor of 5-8. Different types of common lubrication were used for testing as well as dry tests (no lubrication).

Contact Pressure:

Previous tests have shown that an increase in contact pressure decreases the CoF to a certain point. Taylor and Stanton (2010) found that the CoF decreased as contact pressure was increased until 3000 psi, after which, the CoF remained constant. Dolce Et al. (2005)) found similar results. Pressures used by

Taylor and Stanton (2010), were 1500 psi (conforming to WisDOT plans), 3000 (mid-point), and 4500 psi (max pressure allowed by AASHTO).

6.2 Materials Used in Testing

6.2.1 PTFE/Steel Sliding Surfaces

PTFE, which is the acronym for polytetrafluoroethylene, is a fluorocarbon-based polymer that naturally has very low friction properties as well as other properties favorable for use as bridge bearings. There are many different types of PTFE that are varied during manufacturing in order to improve or alter performance in certain areas. Some of these other properties include high resistance to weathering, high and low temperature capability, and high chemical resistance.

The material needs for a lateral bridge slide are different than for a bridge bearing. Low initial friction is a primary concern, while long-term durability is not a concern. Fillers such as glass or carbon fibers as well as many other types of fibers, can be added to the PTFE during the manufacturing process to improve strength and resistance to wear. The PTFE used for this project was "Unfilled" meaning it had no fibers added. This was chosen because it has better (lower) friction properties than if it was reinforced with fiber. The samples were also virgin PTFE. This means that it did not contain any recycled PTFE, which also improves frictional properties.

Both dimpled and smooth PTFE pads were used in the testing. The dimpled pads were used for the tests with lubricants, whereas the smooth pads were used for the dry tests. The dimples are designed to retain some of the lubricant so that while sliding, the PTFE stays lubricated longer. Figure 6.2.1-1 shows both a dimpled and a smooth PTFE pad.

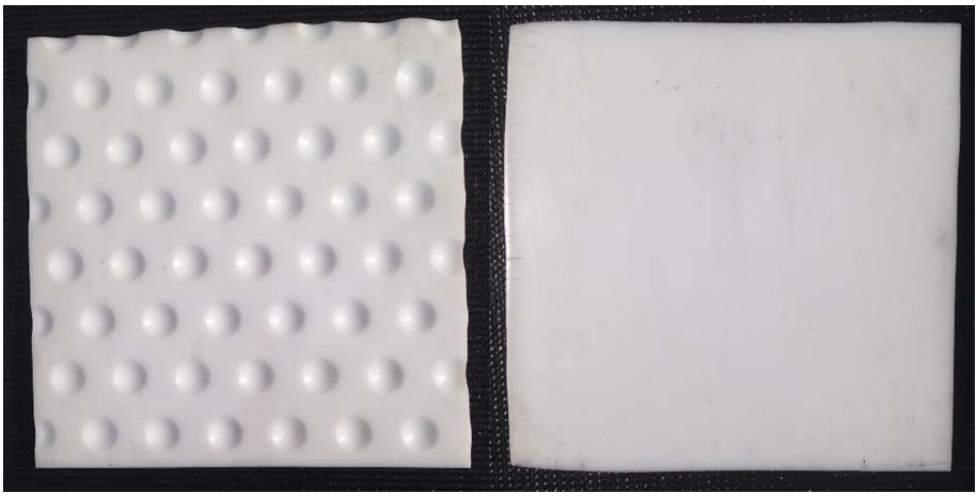


Figure 6.2.1-1 PTFE Pads Used in Testing

The *AASHTO LRFD Bridge Design Specifications* contain specifications for bridge bearings constructed with PTFE sliding surface. The values for CoF indicate that contact pressure has an effect on friction. Based on this the team decided to also vary contact pressure as a testing parameter.

Contact pressure is calculated by finding the portion of the bridge weight concentrated on each PTFE pad and then dividing that load by the surface area of the pad. The team decided to use 4”x 4” PTFE pads that would allow vertical loads to be in a manageable range for the testing apparatus. The *AASHTO LRFD Bridge Design Specifications* do not vary design parameters based on scale; therefore this reduced scale would not impact the outcome of the experiment.

The calculations for contact pressure were based on the maximum contact pressure to be tested on the pads, which was 8000 psi. The 8000 psi value was set at a reasonable maximum value based on discussions with bearing manufacturers. Multiplying this pressure by the area of 16 square inches would lead to a maximum vertical load of 128,000 pounds. This load was determined to be reasonable for the tests and the testing apparatus.

Two companies that manufacture and sell bridge bearings were contacted in order to obtain PTFE samples. The companies, D.S. Brown and R.J. Watson, agreed to donate the PTFE samples. R.J. Watson and D.S. Brown each sent 12 PTFE pads (4 smooth pads and 8 dimpled pads).

6.2.2 Stainless Steel and Carbon Steel

The stainless steel used for the testing was Type 304 stainless which is what is specified for permanent bridge bearings in the *AASHTO LRFD Bridge Design Specifications*. Stainless steels are used almost exclusively in the lateral bridge slide market. This is due to the fact that the slide area is typically left exposed in the completed bridge. The most common surface finish of stainless steel in use is No. 8 “mirror” finish. This is commonly used in high load bridge bearings where low friction is critical.

The bearing manufacturers noted that there are other less expensive stainless steel options in the market that have comparable surface finishes. The goal for this testing was to compare these other stainless steels that have different surface roughness. Figure 6.2.2-1 shows the 2 kinds of stainless steel that were used. Unfinished carbon steel was also tested to investigate its feasibility. The specimen on the left of the photo is stainless steel with a No. 2B finish, and the middle specimen is stainless steel with a No. 1 finish. The specimen on the right of the photo is carbon steel.

There are many other different types of finishes for stainless steel. Each one goes through a different process when being made which gives it a different finish and consequently a different surface roughness. Table 6.2.2-2 shows the available finishes, the grit number, and the processes that they go through to create the finish they have. The No. 2B finish is the most common and least expensive.

Depending on the thickness of the stainless steel, it will either be referred to as plate or sheet stainless steel. If the stainless steel is equal to or thicker than 3/16 inch (0.1875 in.), then it is considered plate stainless steel. If it is less than that thickness (0.1874 in. or less) then it is considered sheet stainless steel. The standard way of measuring sheet stainless steel is in gages. The No. 2B stainless steel sheets that were used were 12 gage (0.105 in thick), and the No. 1 stainless steel as well as the carbon steel plates used were 3/16” thick. The No. 1 stainless steel is only manufactured in plate thicknesses, which is why the stainless was 3/16 in thick for the specimens.

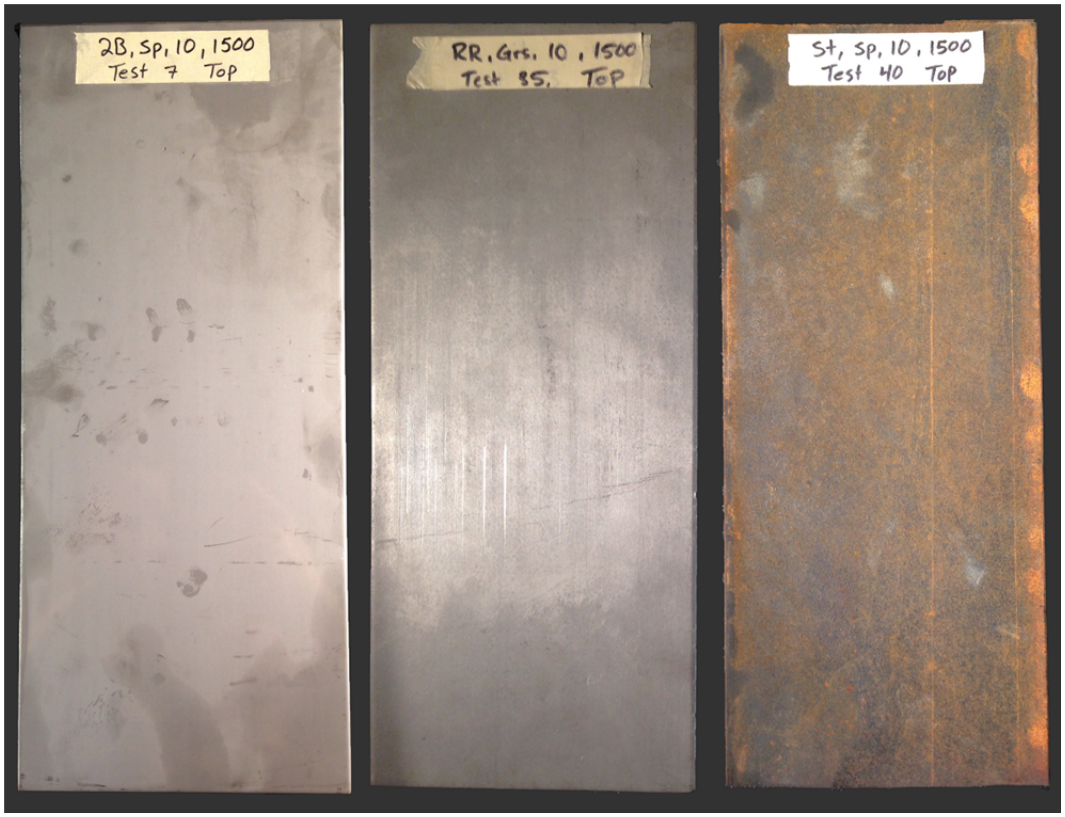


Figure 6.2.2-1 Stainless Steel Used in Testing: Left: #2B Finish Stainle4ss Steel, Center: RR Stainless Steel, Right: Carbon Steel

STAINLESS STEEL FINISHES		
Finish No.	**Grit No.**	**Metallurgy Process**
No. 1 Finish	60	Hot-rolled, annealed and descaled
No. 2D Finish	80	Hot-rolled, annealed and descaled, dull finish
No. 2B Finish	100-320	Cold-rolled, annealed, bright finish
No. 3 Finish	100-120	Intermediate polished finish, usually 120 grit, one or both sides
No. 4 Finish	120-220	General purpose polished finish, usually 180 grit, one or both sides
No. 6 Finish	240	Dull satin finish, Tampico brushed, one or both sides
No. 7 Finish	320	High luster finish or near mirror (has some lines)
No. 8 Finish	400-500	Mirror finish

Table 6.2.2-2 Available Stainless Steel Finishes

There were several important differences between the sheets of the No. 2B stainless steel sheets there were purchased. The first order of No. 2B stainless was for sheets that were 8” x 18”. Some of these sheets were scratched during transport. Several sheets with bad gouges were not used for testing due to the concerns regarding the potential of higher CoF values. Several sheets had light scratches that still felt

smooth to the touch were still used for the first 3 No. 2B tests. More No. 2B stainless steel sheets were purchased and used in the subsequent tests.

The first order of No. 2B stainless steel sheets seemed to be slightly rougher to the touch when compared to the second order of No. 2B sheets (which also had minor scratches). These sheets were used for the first 3 tests (1500, 3000 and 4500 psi using 2B stainless, at 10 in/min, and no lubrication (dry)). The results for these test produced slightly higher CoF values than expected which will be discussed further in the results section of this report.

6.2.3 Lubricants

Lubricants can be used on PTFE sheet sliding systems. They are not common in bridge bearings due to the need to re-apply them at regular intervals. This make lubricated bearings unpopular in the bridge market due to higher maintenance costs. Lubricants are feasible for lateral bridge slides due to the short duration nature of the work.

Several lubricants were used for the testing. The lubricants chosen and the reasons for their selection are as follows:

1. Dishwashing soap: Commonly used in lateral slide projects.
2. Silicone grease: Commonly used in bridge bearings.
3. Motor oil: A readily available and inexpensive lubricant.
4. Graphite: Commonly used lubricant.

The soap that was used as a lubricant was liquid Dawn dish washing soap (which happens to be commonly used in lateral slide projects for no specific reason). The oil that was used was 10W-40 Motor Oil. The grease used was SAE-AS8660 dielectric grease made by Jet-Lube. SAE-AS8660 is what AASHTO specifies when using lubricants on PTFE sliding bearings for regular bridge bearings. The graphite used was just a standard powdered graphite purchased at a local hardware store. Each of these lubricants is shown in Figure 6.2.3-1 below.

Figure 6.2.3-1 Lubricants Used in Testing

6.2.4 Steel Substrate

Steel substrates were used to confine the PTFE pads to help prevent them from deforming as a contact pressure was applied. An area of 4" x 4" was machined into the steel plates where the PTFE pads could sit. The recess was machined to a depth of 3/32 of an inch. Once the PTFE pads were inserted into the recess, half of the pad would protrude above the recess since the PTFE was 3/16 inch thick. Most of the PTFE pads fit very snug in the substrate. A few were loose; therefore a thin shim was added to produce a tight fit.

6.3 Test Set-Up

The testing was done in a structures lab at USU in Logan, Utah. The lab has a reaction frame in which the test set-up was mounted. Figure 6.3-1 is a sketch of how the team planned on using this reaction frame to attach the components. Two vertical hydraulic rams were used to apply a compression force into the pad, simulating the vertical load caused by the weight of a bridge. A spreader beam was used between the rams to provide uniform load during the slide testing. Two samples were inserted between the spreader beam and the foundation. A horizontal actuator was then used to push the bearing back and forth, simulating a bridge slide.

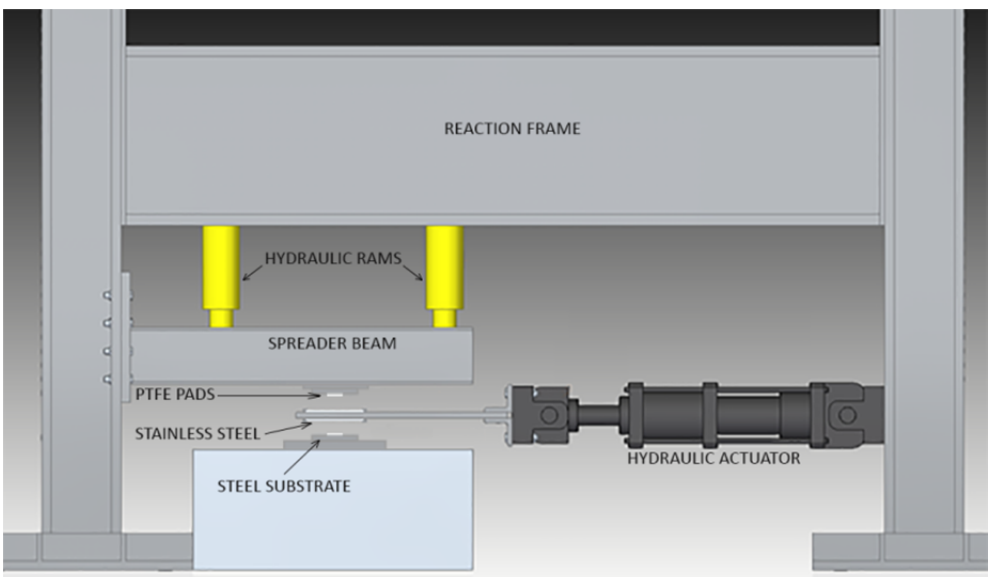


Figure 6.3-1 Sketch of Testing Apparatus

It was necessary to secure the spreader beam to the reaction frame In order to keep the spreader beam from moving when the actuator slid the bearing. Slotted holes were cut into the end plate of the spreader beam so that beam would move freely while the vertical load was being applied. The bolts were installed hand tight to take out any play in the joint, but still loose enough to prevent them from carrying any load. Figure 6.3-2 shows a zoomed in area of the slots in the end plate of the spreader beam.

Figure 6.3-2 Slotted End Plate of Spreader Beam

When using sliding bearings in a lateral bridge slide, it is common to place the PTFE bearing pads within a slide track or on top of the abutment with the PTFE surface facing up. The bearings support the bridge via a stainless steel slide shoe cast into the underside of the superstructure. The set-up in the lab was somewhat different in that there needed to be 2 PTFE pads and 2 stainless steel plates used in order for the system to work properly and not have any extra resistance from the use of a roller or some other type of set-up. This meant that the friction would be doubled since there were 2 sliding surfaces. This was accounted for in calculating the CoF and will be discussed later in this report.

Initially the plan was to attach the PTFE pads to the steel tongue and push them back and forth sliding along the stainless steel plates. This would have created a slightly eccentric loading on the pads as they slid from one end of the stroke to the other. To avoid this issue it was decided to keep the pads stationary by attaching them to the spreader beam above and the concrete block below. The stainless steel plates would then be attached to both sides of the steel tongue and slid back and forth.

Figure 6.3-3 shows how the stainless steel and PTFE were attached and how they were sandwiched together. The steel substrates used to confine and hold the PTFE were tack welded to the spreader beam and large plates below on the concrete block. The friction between the large steel plates and the concrete block was always higher than the friction between the PTFE pads and stainless steel, which kept the base plate from sliding on top of the concrete block.

Figure 6.3-3 Testing Sample Set-Up

Load cells were placed between the vertical rams and the spreader beam to measure the vertical load for each ram. Figure 6.3-4, which shows the full test set-up. The horizontal actuator used in the tests is a servo hydraulic ram made by MTS, which has a load cell built into it. The MTS ram also had the ability to measure the displacement as it moved back and forth as well as the speed, and the number of cycles.

Figure 6.3-4 Full Test Set-Up

6.4 Tests Completed

During the previous research on testing parameters for PTFE bearings, certain parameters influences the CoF were identified. Of the 14 parameters found during the literature search, 4 of the most influential ones were chosen to be tested for this project. The following test matrix shown in Table 6.4-1 shows the tests that were completed. Each test is represented by a code that represents the parameters that were used for that specific test. Table 6.4-2 contains a key for what each of the abbreviations used in these codes represents:

Test Matrix		Lubricants Used between PTFE and Stainless Steel Surfaces				
Slide Speed (in/min)	Pressure (psi)	Dry Smooth(sm) or Dimpled (dim)	Dish Soap	Silicone Grease (SAE-AS 8660)	Automotive Motor Oil	Graphite
No. 2B Stainless Steel						
10 in/min	1500	2B,D-sm,10,1500	2B,Sp,10,1500 †	2B,Grs,10,1500	2B,Oil,10,1500	
	3000	2B,D-sm,10,3000	2B,Sp,10,3000	2B,Grs,10,3000	2B,Oil,10,3000	
	4500	2B,D-sm,10,4500	2B,Sp,10,4500 †	2B,Grs,10,4500	2B,Oil,10,4500	2B,Gph,10,4500
	8000	2B,D-dim,10,8000	2B,Sp,10,8000	2B,Grs,10,8000	2B,Oil,10,8000	
2.5 in/min	1500	2B,D-dim,2.5,1500	2B,Sp,2.5,1500	2B,Grs,2.5,3000‡		
	3000	2B,D-dim,2.5,3000	2B,Sp,2.5,3000	2B,Grs,2.5,3000		
	4500	2B,D-dim,2.5,4500	2B,Sp,2.5,4500	2B,Grs,2.5,4500	2B,Oil,2.5,4500	2B,Gph,2.5,4500
	8000					
No. 1 Stainless Steel						
10 in/min	1500	RR,D-sm,10,1500	RR,Sp,10,1500	RR,Grs,10,1500	RR,Oil,10,1500	
	3000					
	4500	RR,D-dim,10,4500	RR,Sp,10,4500	RR,Grs,10,4500	RR,Oil,10,4500	*
	8000					
2.5 in/min	1500	RR,D-sm,2.5,1500	*	*		
	3000					
	4500	RR,D-sm,2.5,4500	*	*	*	*
	8000					
Carbon Steel						
10 in/min	1500	*	St,Sp,10,1500	St,Grs,10,1500	St,Oil,10,1500	
	3000					
	4500	*	St,Sp,10,4500	St,Grs,10,4500	St,Oil,10,4500	*
	8000					
2.5 in/min	1500	*	*	*		
	3000					
	4500	*	*	*	*	*
	8000					

* Tests considered before testing began, but were not done based on results of prior tests.
† Additional tests were done using Soap and adding water during testing to simulate effects of rain.
‡ This test was done at the wrong pressure. 3000 Psi was used instead of the 1500 psi.

Table 6.4-1 Test Matrix

2B	Stainless steel mating surface with a No. 2B surface finish
RR	Stainless steel mating surface with a No. 1 surface finish which is also known as rough-rolled (RR) stainless steel
St	Carbon steel mating surface
D-sm	Dry test (no lubrication), with smooth (sm) PTFE pads
D-dim	Dry test (no lubrication), with dimpled (dim) PTFE pads
Sp	Soap lubrication
Grs	Grease lubrication
Oil	Oil lubrication
Gph	Graphite lubrication
10	A sliding speed of 10 in/min
2.5	A sliding speed of 2.5 in/min
1500	Contact pressure of 1500 psi
3000	Contact pressure of 3000 psi
4500	Contact pressure of 4500 psi
8000	Contact pressure of 8000 psi

Table 6.4-2 Terms of Test Codes Used to Identify Each Test Described

6.4.1 Data Acquisition and Data Processing

Two acquisition units were used to collect the data. The MTS ram had its own system. The 2 vertical rams were connected to a separate system. The vertical data acquisition system only measured the vertical force for both vertical rams and time. The MTS data acquisition was set up to measure the horizontal forces, horizontal displacement, cycle count, and time. The sampling rate was set at 100 samples/second to make sure that the spikes in force as seen in Figure 6.4.1-1 could be captured.

The sliding speed was set to be constant at either 10 in./min or 2.5 in/min depending on the test. The ram was set to push the samples back and forth ± 1 inch, corresponding to a stroke of 2 inches. In order to produce a sliding speed that was constant, the MTS actuator was programed to run a ramp waveform, instead of a sinewave or square wave.

As the ram would reach the end of the stroke it would switch directions and start moving the other way. When this change in direction happened the force required to push it would spike higher due to static breakaway friction. Figure 6.4.1-1 shows a graph of the horizontal force on the y-axis and displacement on the x-axis. This graph shows all 101 cycles for the test. The highest spike in force on the left hand side is the static friction for the first cycle. The force decreased significantly after the first cycle, and then slowly continued to decrease as the cycles (or slide path) increased.

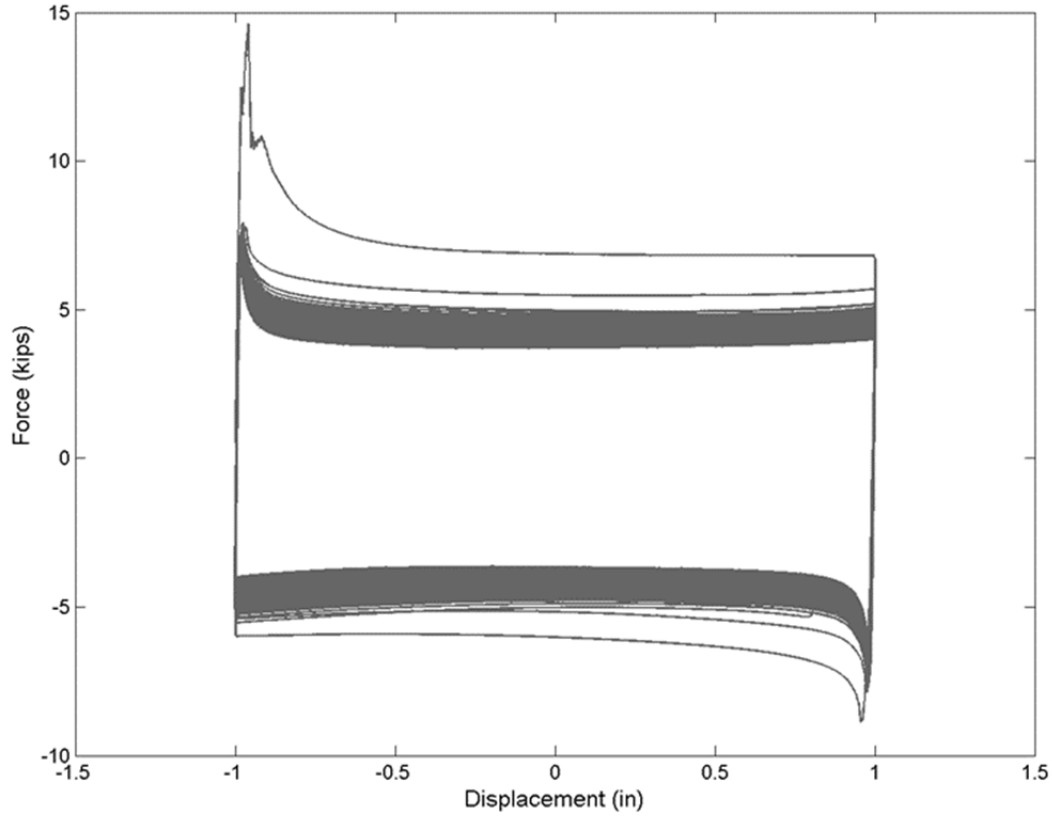


Figure 6.4.1-1 Typical Graph of Force Versus Displacement for All 101 Cycles

It is easier to see the trend of the force decreasing over time when the cycles are spread out on a force versus time plot. Figure 6.4.1-2 shows the first 12 cycles of a test. The static breakaway friction is highest on the first cycle, and then decreases slowly for each progressing cycle.

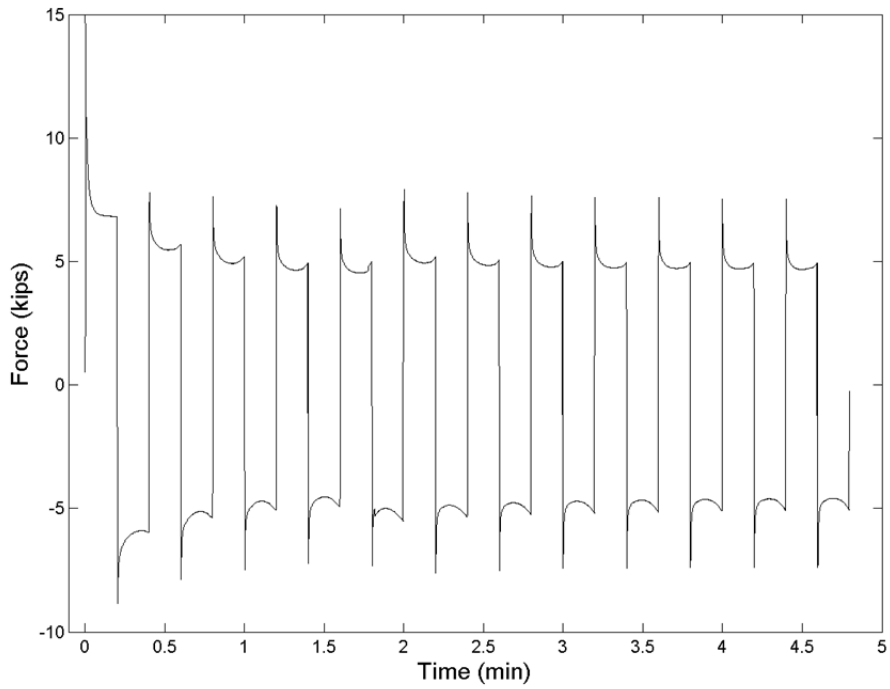


Figure 6.4.1-2 Typical Graph of Force Versus Time (Cycles 1–12)

The CoF can be calculated using a simple equation relating the vertical force (or normal force (N)) to the horizontal force (or friction force (F)),

$$F = \mu * N \qquad \text{(Eq. 6.4.1-1)}$$

Where:
F = Friction force
μ = CoF
N = Normal force

Since the set-up used had 2 sliding surfaces there are 2 friction forces so the proper equation for this project then becomes

$$F = 2 * \mu * N \qquad \text{(Eq. 6.4.1-2)}$$

Rearranging the equation to solve for the CoF gives the equation in this form:

$$\mu = \frac{F}{2 * N}, \qquad COF = \frac{Horizontal\ Force}{2 * Vertical\ Force} \qquad \text{(Eq. 6.4.1-3)}$$

Once the CoF was found for each test, graphs were created to easily compare and analyze the data. When the CoF is plotted, as shown below in Figure 6.4.1-3, it looks much like the last plot of force vs time but with different values.

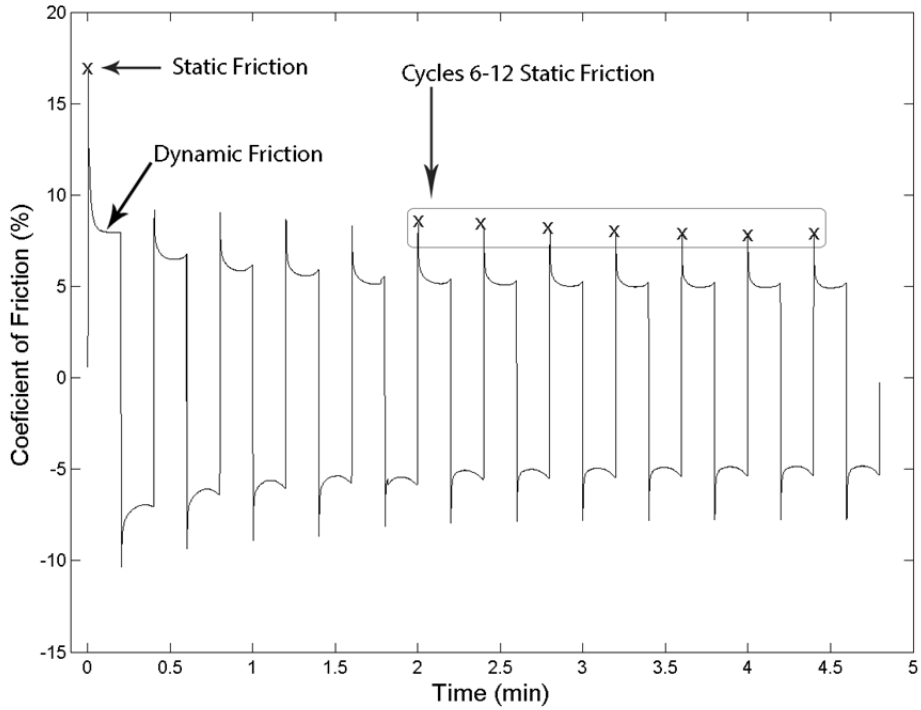


Figure 6.4.1-3 Typical Graph of CoF Versus Time (Cycles 1–12)

There are 2 main types of friction in this type of system: static (or breakaway) friction and dynamic friction. The static friction is the amount of friction that keeps an object from moving. In order for an object to start sliding it has to overcome the static friction force. Dynamic friction is the friction present once the object is in motion. The dynamic friction has historically always been lower than the static friction for common materials. This was confirmed with this testing; therefore the values for the static friction are what will be focused on in the analysis of the test results.

In order to summarize the behavior of each test, 3 important values have been recorded in Table 6.4.1-4. The 3 values are the static friction for cycle 1, the average of cycles 6-12, and cycle 101. Figure 6.4.1-3 shows static friction for cycle 1 and also cycles 6-12. It should be noted that this plot does not show the point at the 101st cycle. The CoF values for all tests are shown in Table 6.4.1-4. A discussion on the behavior of each test is included in the following sections.

STATIC COEFFICIENT OF FRICTION (%)																		
Slide Speed	10 in/min									2.5 in/min								
Roughness	**2B Stainless**			**RR Stainless**			**Carbon Steel**			**2B Stainless**			**RR Stainless**			**Carbon Steel**		
Cycle	*1*	*6-12*	*101*	*1*	*6-12*	*101*	*1*	*6-12*	*101*	*1*	*6-12*	*101*	*1*	*6-12*	*101*	*1*	*6-12*	*101*
1500 Psi																		
Dry	7.75 *	3.24 *	3.03 *	14.12	6.53	5.85	-	-	-	4.23	4.24	3.16	-	-	-	-	-	-
Soap	5.66	2.28	2.84	24.25	6.99	4.05	18.35	7.63	4.63	4.50	1.90	2.90	-	-	-	-	-	-
Grease	5.15	0.99	1.55	4.20	2.81	3.26	6.86	4.30	3.81	-	-	-	-	-	-	-	-	-
Oil	4.04	1.05	1.49	16.07	5.92	3.10	20.15	6.22	4.32	-	-	-	-	-	-	-	-	-
Graphite	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3000 Psi																		
Dry	16.76 †	7.93 †	6.81 †	-	-	-	-	-	-	2.57	2.11	2.27	-	-	-	-	-	-
Soap	2.18	1.27	1.87	-	-	-	-	-	-	5.07 ‡	1.85 ‡	1.97 ‡	-	-	-	-	-	-
Grease	2.02	0.55	1.11	-	-	-	-	-	-	1.86	0.50	1.12	-	-	-	-	-	-
Oil	1.51	0.87	1.09	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Graphite	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
4500 Psi																		
Dry	12.1 †	4.90 †	4.75 †	4.10	2.47	2.33	-	-	-	2.99	1.42	1.89	-	-	-	-	-	-
Soap	2.45	1.20	1.57	5.02	3.97	3.49	7.05	3.27	2.01	0.70	0.50	0.84	-	-	-	-	-	-
Grease	1.06	0.46	0.81	4.11	2.28	2.92	2.75	1.73	1.72	0.48	0.40	1.01	-	-	-	-	-	-
Oil	2.75	0.77	0.79	5.99	2.60	1.47	6.85	3.19	1.86	1.08	0.68	0.74	-	-	-	-	-	-
Graphite	9.49	9.67	6.12	-	-	-	-	-	-	6.16	5.85	6.11	-	-	-	-	-	-
8000 Psi																		
Dry	2.94	0.88	0.69	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Soap	1.44	0.88	1.26	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Grease	0.67	0.32	0.47	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Oil	0.90	0.46	0.47	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Graphite	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

* Stainless steel previously used for a grease test was used for this test. Small remnants of grease remained which reduced the friction
† Tests used 2B stainless steel sheets that were a little rougher than the others, which is likely to have increased the friction slightly
‡ 3000 psi was applied to the PTFE pads for 32 hours before testing to see if static COF for cycle 1 would increase after being loaded for a long time

Notes: Cycle 1 equates to first cycle.
Cycles 6–12 refers to the mean of cycles 6–12.
Cycle 101 refers to cycle 101.

Table 6.4.1-4 Static CoF Values for All Tests

6.4.2 Test Results

6.4.2.1 Dry Tests

Dry tests were completed using the No. 2B stainless steel as well as the No. 1 stainless steel. There were no dry tests completed on the carbon steel based on the assumption that the CoF would be higher than desirable. Three of the dry tests were completed with the No. 2B stainless steel that had a slightly rougher surface than the remainder of the No. 2B sheets. Figure 6.4.2.1-1 is a picture of one of the dry tests that used the smoother No. 2B stainless steel sheets. Both dimpled and smooth PTFE pads were used for the dry tests in order to determine if there would be any differences. This is indicated in the test matrix shown in Table 6.4-1.

Figure 6.4.2.1-1 Dry Test Which Used Dimpled PTFE Instead of Smooth

6.4.2.2 Soap Tests

Liquid dish soap was applied to both the PTFE pads as well as the stainless steel or steel plates. Dimpled PTFE pads were used for all of the soap tests. Figure 2.4.2.2-1 is a typical picture showing the application of soap between the PTFE and stainless steel or steel. Figure 2.4.2.2-2 shows a close-up view of the PTFE pad and how thick the layer of soap typically was.

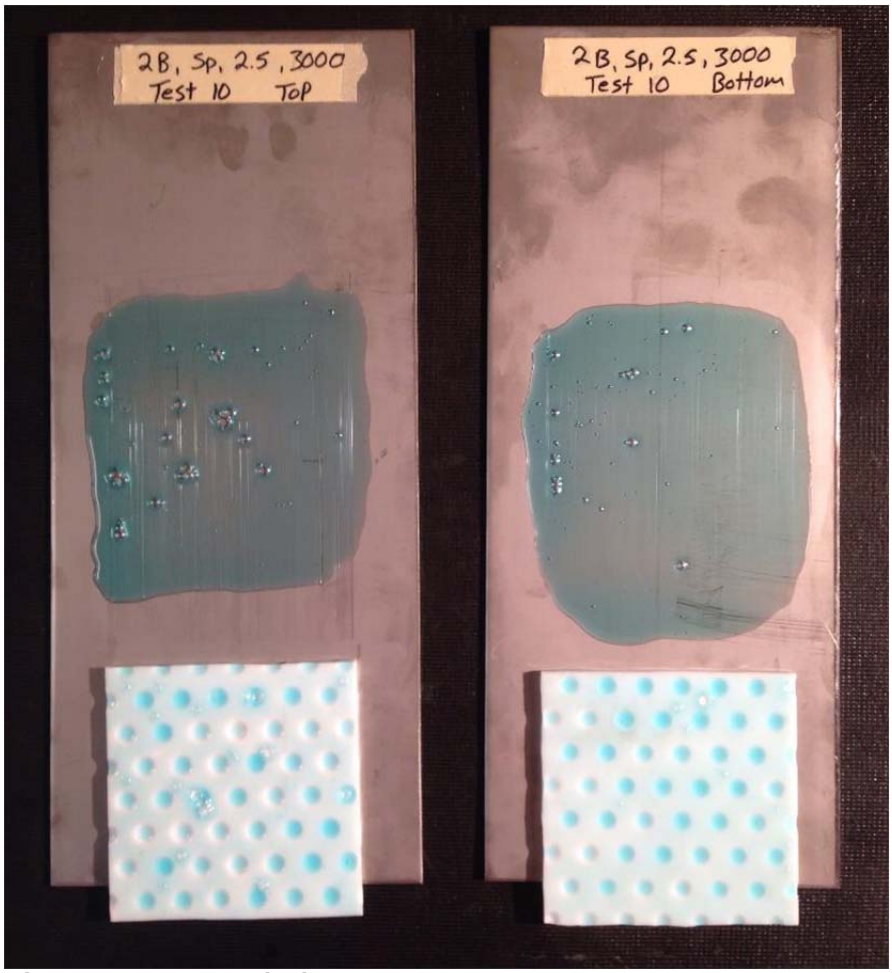


Figure 6.4.2.2-1 Typical Soap Test

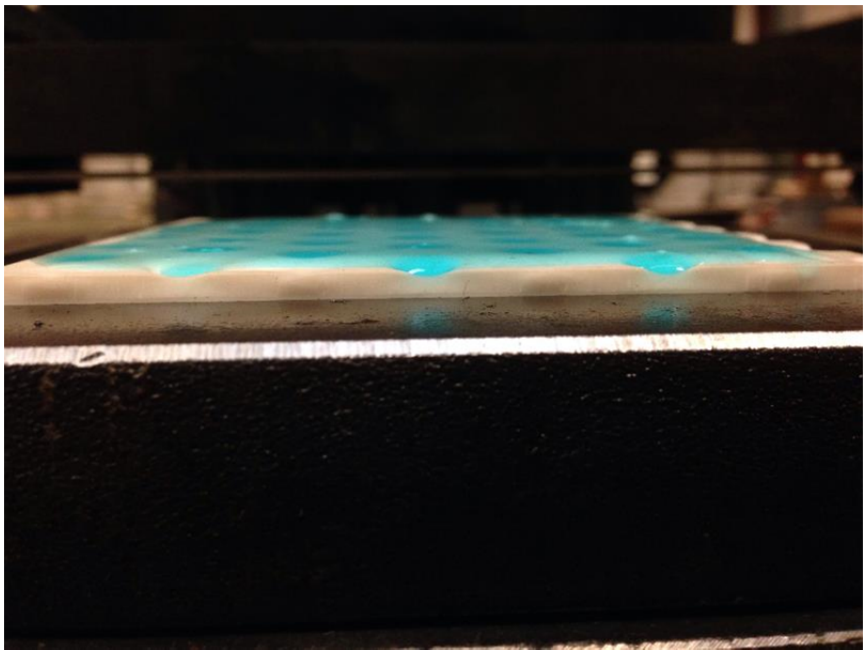


Figure 6.4.2.2-2 Typical Soap Test Shows the Layer of Dish Soap on a Dimpled PTFE Pad

6.4.2.3 Grease Tests

The type of grease that was used was chosen based on the AASHTO LRFD Bridge Design Specifications for permanent bridge bearings. The grease is SAE-AS8660, which is silicone based dielectric grease. Dimpled PTFE pads were used for all of the grease tests. A thick layer of grease was applied to the PTFE pads so that all of the dimples were filled and there was an even layer of grease on the whole pad.

A layer of grease was also applied to the stainless steel and carbon steel plates. Figure 6.4.2.3-1 shows the before picture for one of the grease tests, and Figure 6.4.2.3-2 shows the same specimens after the test was complete. Even though the majority of the grease would get pushed off to the side, the sliding surfaces stayed lubricated throughout the tests.

Figure 6.4.2.3-1 Typical Grease Test Grease Was Applied to PTFE Pads and Stainless Steel or Steel Plates

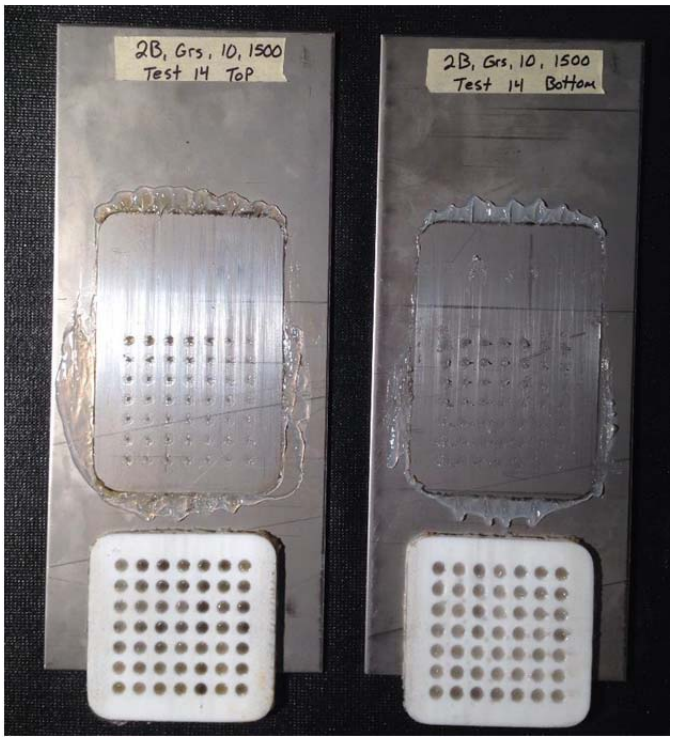


Figure 6.4.2.3-2 Typical Grease Test After Testing

6.4.2.4 Oil Tests

The type of oil that was used for the testing was 10W-40 automotive engine oil. An adequate layer of oil was applied to the specimens. The interface surface stayed well lubricated throughout the testing. Figure 6.4.2.4-1 shows the amount of oil used on the tests and the type of oil used.

Figure 6.4.2.4-1 Typical Oil Test Showing Oil on PTFE and Stainless Steel Before Testing

6.4.2.5 Graphite Tests

Standard tubes of powdered graphite were purchased from a local hardware store. Figure 6.4.2.5-1 shows a picture of one of the tubes that was used for the graphite tests. Both the PTFE and the stainless steel were coated with the graphite for the test.

For the first graphite tests, a liberal amount of graphite was placed on the pad, based on the assumption that the majority of the graphite would be pushed off during the beginning of the test, leaving an adequate layer of graphite left on the sliding surface for the remainder of the test. The results of this test were very poor. The friction force was very high and the PTFE pads deformed significantly due to excessive heat caused by the high friction. It was suspected that there was too much graphite put on the test specimens and that using smooth PTFE instead of dimpled would possibly give better results. Figure 6.4.2.5-2 shows how much graphite was used for the first test. This test was done using dimpled PTFE pads.

Figure 6.4.2.5-1 Graphite Powder Used in Testing

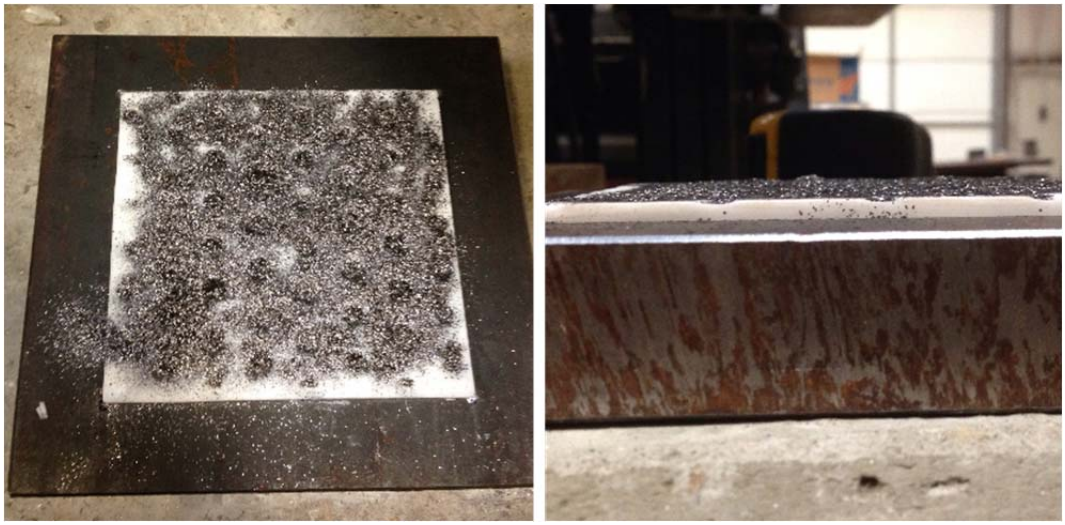


Figure 6.4.2.5-2 Test Specimen with Graphite

Figure 6.4.2.5-3 shows the deformation of the PTFE pad including curling out from under the steel substrate. At the end of the tests the steel substrate holding the PTFE and the steel tongue holding the stainless steel, were hot to the touch. Figure 6.4.2.5-4 illustrates how much the PTFE pad deformed. Half of the thickness of the pad was confined by the steel substrate which is why part of the pad retained its square shape as shown on the right side of Figure 6.4.2.5-4. This was the original size and shape of the pad.

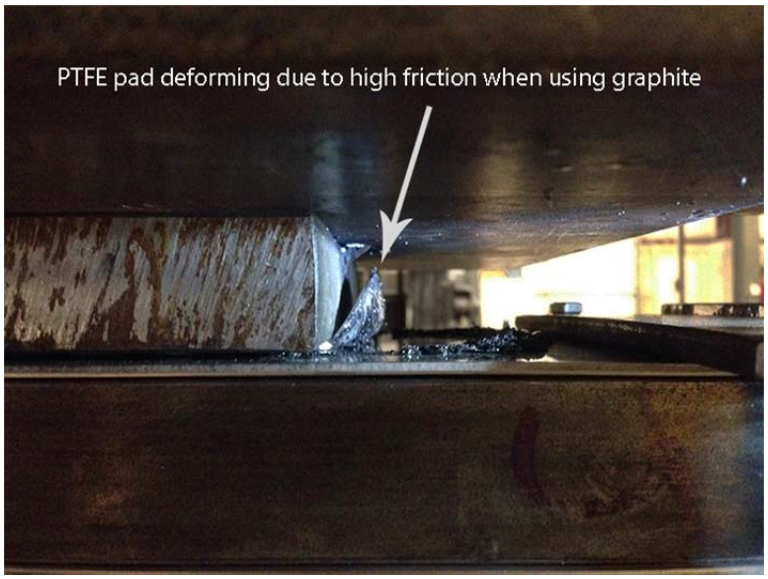


Figure 6.4.2.5-3 Deformed PTFE Pad with Graphite Lubricant

Unconfined side Confined side

Figure 6.4.2.5-4: Deformed PTFE Pad After Testing with Graphite Lubricant

For the second graphite test, smooth PTFE pads were used in place of the dimpled pads and only a light even coating of graphite was applied in hopes to get lower CoF values than with the first test. Figure 6.4.2.5-5 shows the stainless steel and PTFE pad used on top. This shows the before and after pictures of the same specimens. The pad did not deform nearly as much as the test with the dimpled PTFE and excessive graphite. This test resulted in a lower CoF than the previous test.

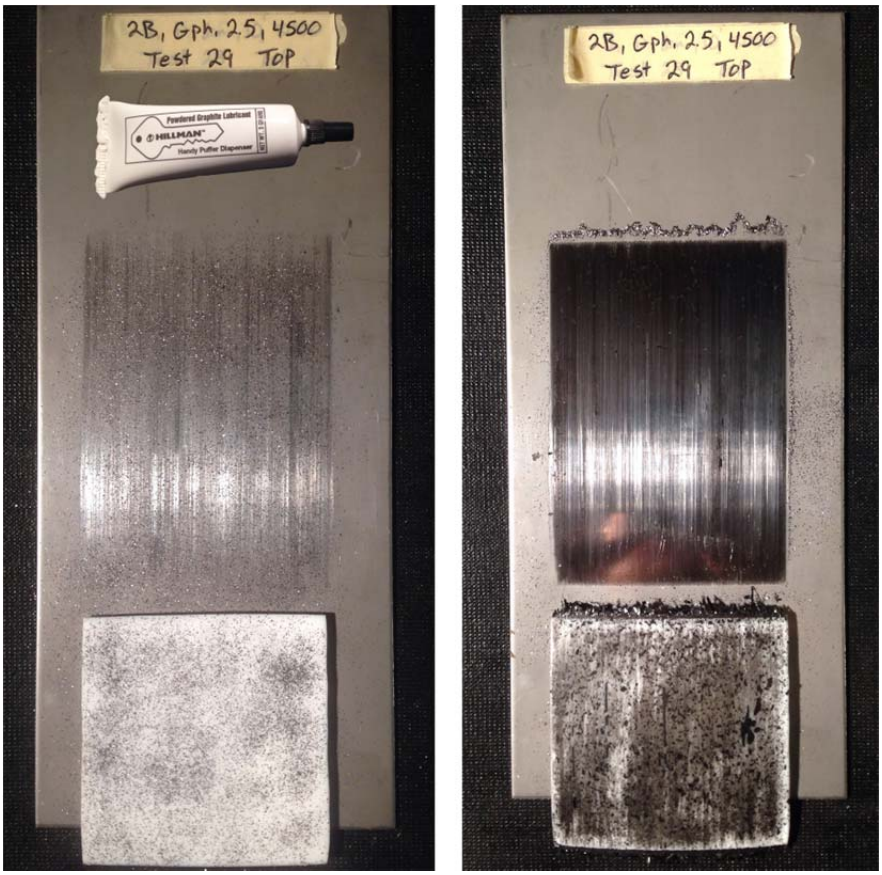


Figure 6.4.2.5-5 Stainless Steel and PTFE Pad Before and After Graphite Test

The performance of both of these graphite tests were considered unsatisfactory when compared to the other test, therefore only 2 tests were completed using the graphite.

6.5 Analysis and Results

When comparing results of the different tests, it is important to only vary one parameter at a time in order to accurately assess how that parameter influences the CoF. Each of the 4 parameters will be evaluated in this manner by referring to the CoF values in Table 6.4.1-4. The parameters will be discussed in the following order: (1) Contact Pressure, (2) Lubricants, (3) Surface Roughness, and (4) Sliding Speed.

6.5.1 Contact Pressure

Upon examination of the influence of contact pressure on the CoF a pattern was found that indicates that as the contact pressure increases, the CoF decreases. Even though this may seem counterintuitive for most, this pattern has been found by other researchers in the past that have investigated the CoF for PTFE sliding surfaces. Examples include work completed by Taylor and Stanton (2010) as well as Campbell and Manning (1990).

The general pattern for this can be seen by comparing tests that vary with pressure but use the same lubricant, sliding speed, and surface roughness as demonstrated in the 3 graphs below which are groupings tests for soap, grease, and oil respectively. Each of these groups of tests used 2B stainless, and was at a sliding speed of 10 in/min. These graphs show the static CoF on the y-axis, and cycles 1-12 and cycle 101 on the x-axis. The values used in these plots are the same values shown in Table 6.4.1-4. This type of graph allows comparison of the static friction for the first cycle, how it behaves right after the first cycle, and finally whether it increases or decreases as the cycle progresses to cycle 101.

Figure 6.5.1-1 compares the contact pressure for the soap tests, Figure 6.5.1-2 compares the tests that used grease, and Figure 6.5.1-3 compares the oil tests. Each of these graphs consistently demonstrates that the friction decreases as the pressure is increases. Note that in Figure 6.5.1-1, either the 3000 psi test is lower than expected, or the 4500 psi test is higher than expected for the first several cycles but then the lines quickly cross so that the CoF for the 3000 psi tests is higher than the one for the 4500 psi. It appears that there is something that caused the first cycle to be off for one of these tests, but the team could not determine the cause.

A similar thing happens with the oil tests, but for these tests the cause is known. Several of the PTFE samples were re-used since there was no damage to the pad. In general, re-used samples perform somewhat better in the initial part of the test. This “break in” of the pad is what led to the variation in the CoF on the initial part of the test.

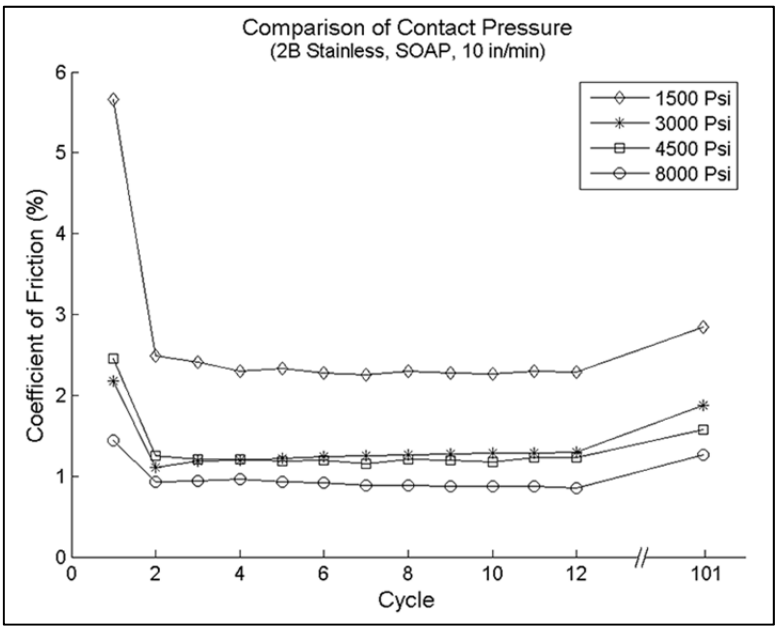


Figure 6.5.1-1 Soap Lubricant Test Results

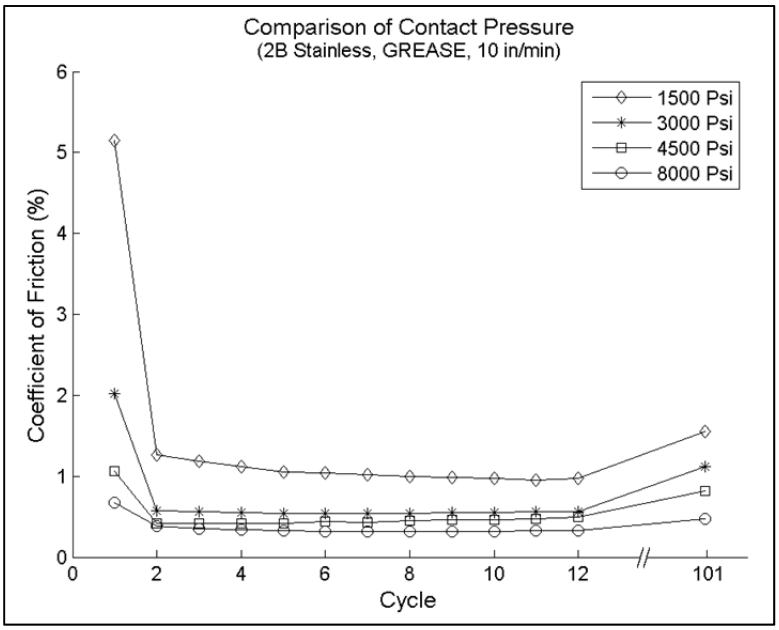


Figure 6.5.1-2 Grease Lubricant Test Results

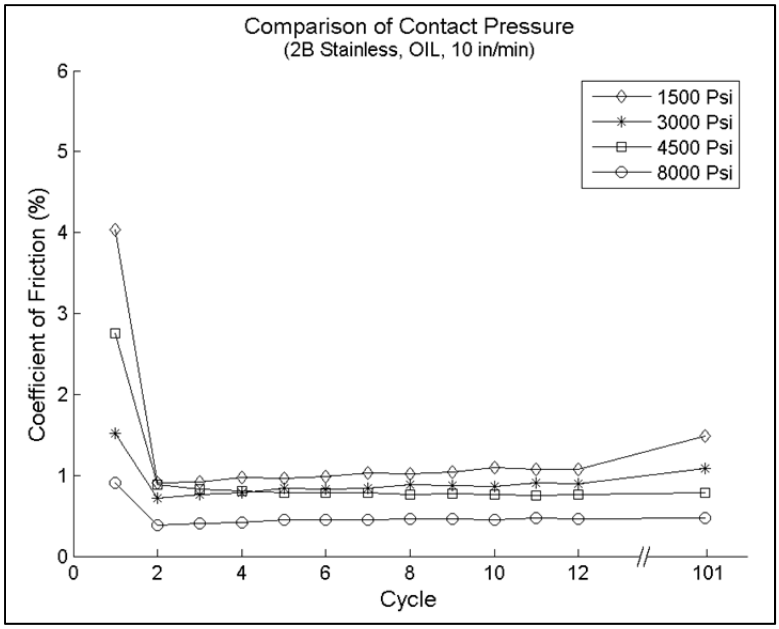


Figure 6.5.1-3 Oil Lubricant Test Results

The group of dry tests that goes with the 3 plots above (using No. 2B stainless steel and 10in/min) follows the same pattern and shows a bigger difference in the CoF between the pressures. These tests are shown in Figure 6.5.1-4. Note that this plot also has a test that seems to be off. The 1500 psi test in this group has a lower CoF than the 3000 psi and 4500 psi test. Being the first test out of the 47 tests completed, there was an issue with the way the data acquisition was set up for this test, and this test needed to be redone. This issue was unknown until most of the other tests had been completed and so it wasn't redone until later.

Because of this the stainless steel that was used for the tests was a pair of sheets that had been used for 3 other tests that used grease. When deciding to use this pair it was assumed that the grease could be cleaned off properly and it would perform similar to the other dry tests that used the No. 2B stainless steel. However, after the test had been completed and the specimens taken out of the set-up, it was obvious those remnants of grease had remained. This little bit of grease caused the friction to be lower than if they were completely dry with no remnants of grease on them. This is why all of the friction values for the 1500 psi test are lower than the ones for the 3000 psi and 4500 psi tests.

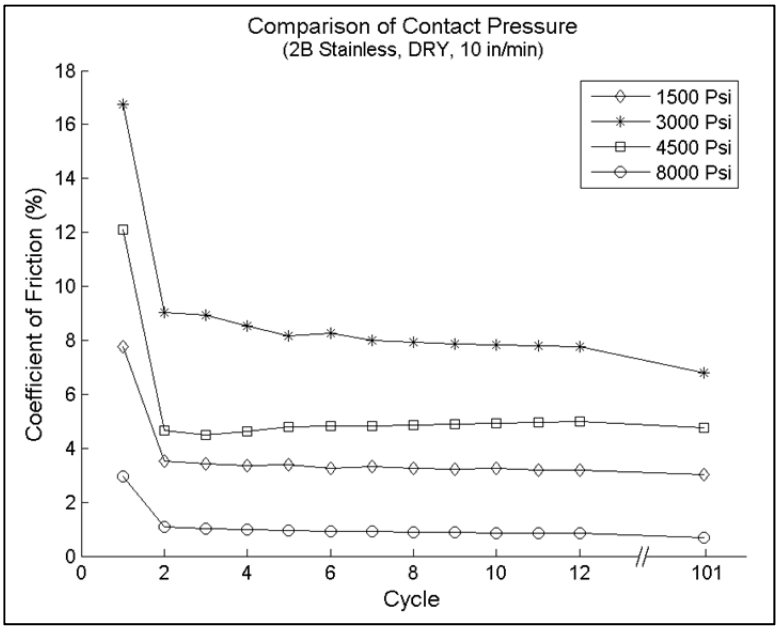


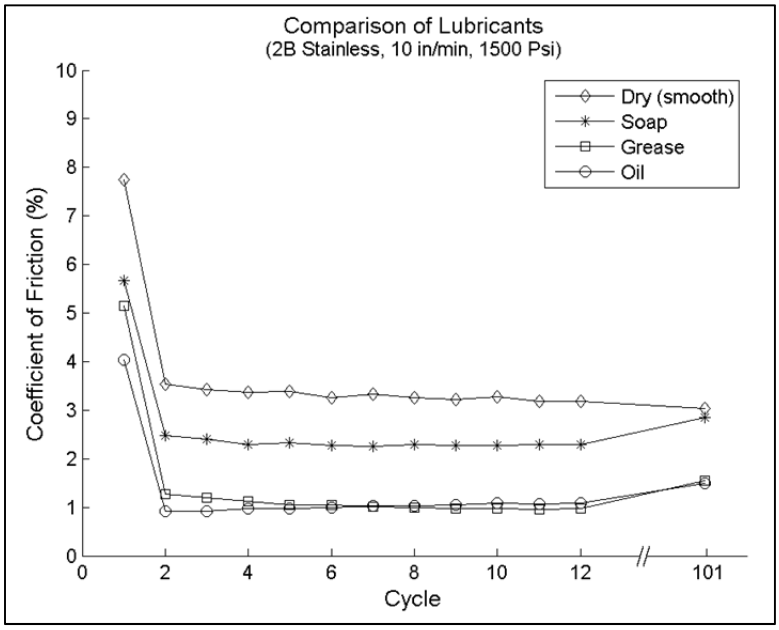
Figure 6.5.1-4 Dry – No Lubricant Test Results

In summary, the test results obtained for this project agrees with other researchers in that the CoF decreases as pressure increases. There were several tests that did not follow this pattern, but the team was able to identify the reasons for these discrepancies. The overall results of all the tests do follow the pattern of decreasing CoF with load.

6.5.2 Lubrication

When using lubricants in combination with dimpled PTFE pads, friction values are significantly reduced, with exception of graphite. The performance of the various lubricants relative to each other is compared in Table 6.1.4-4. A review of the table indicates that the dry tests and graphite tests produce the highest CoF values. Soap performed better than the dry tests overall, and grease and oil performed the best. Note that there are several tests that have numbers that do not match the pattern of the others. There are brief explanations for this behavior in the footnotes of Table 6.4.1-4.

To help visualize the behavior of different lubricants, several plots were developed. They are shown below with different groupings of tests, similar to the review of contact pressure above. The first group of tests includes all of the tests that used a 2B surface roughness, a sliding speed of 10 in/min, and a contact pressure of 1500 psi. Figure 6.5.2-1 shows a comparison of the CoF values for these tests. Again, this plot shows cycles 1-12, and cycle 101.



Constant parameters: 2B, 10 in/min, 1500 psi

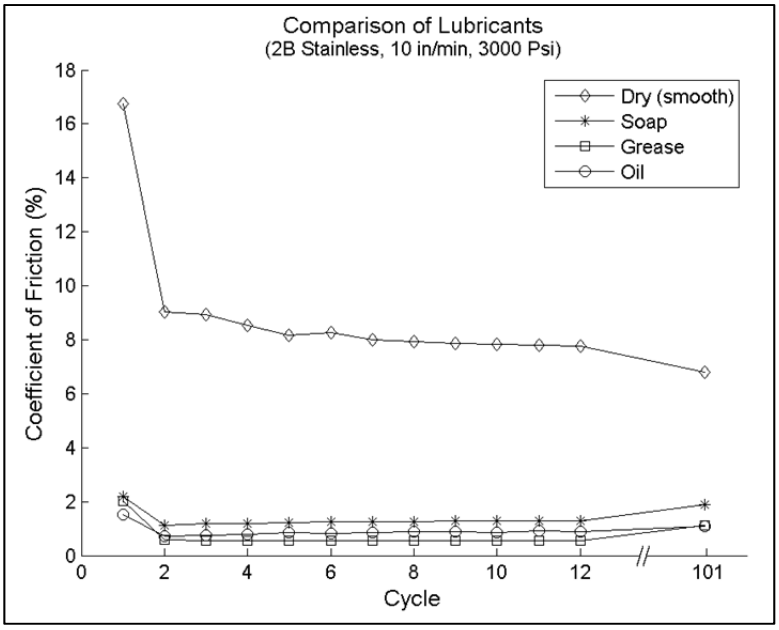
Figure 6.5.2-1: Comparison of CoF for Different Lubricants

The grease and oil tests performed similarly and produced much lower CoF values when compared to the dry test and the soap test. The points at cycle 1 are the most important values in this group of tests since they are the highest points for the plots. Looking at the first cycle, oil performed 50 percent better than the dry test. It dropped to about 1% for the subsequent cycles, and then only increasing slightly to 1.5% at the end of the test. The specimens remained well lubricated throughout the tests.

The grease test had the second best result with a CoF value of 5.15% for cycle 1, then dropping to approximately 1%, and then slightly increasing as the slide path increased. The soap started with a 5.7% CoF for cycle 1, and dropped to an average of 2.3%, and then increased to just below 3% at cycle 101.

The dry test had a CoF that started at 7.75% for cycle 1, then dropped to around 3.5% for cycles 2-12. However, unlike the lubricated tests, the friction for the dry test actually decreased as the slide path increased. This is due to the PTFE pads wearing and becoming smoother.

After comparing the tests at a pressure of 1500 psi, the next group of tests compared was at a pressure of 3000 psi load. Figure 6.5.2-2 plots the CoF for this group. Each of the tests used No. 2B stainless steel, a sliding speed of 10 in/min, and 3000 psi. As noted previously, the dry test for in this group used a stainless steel sheet that had a minor gouge across the surface that is the likely cause of the raised the CoF.

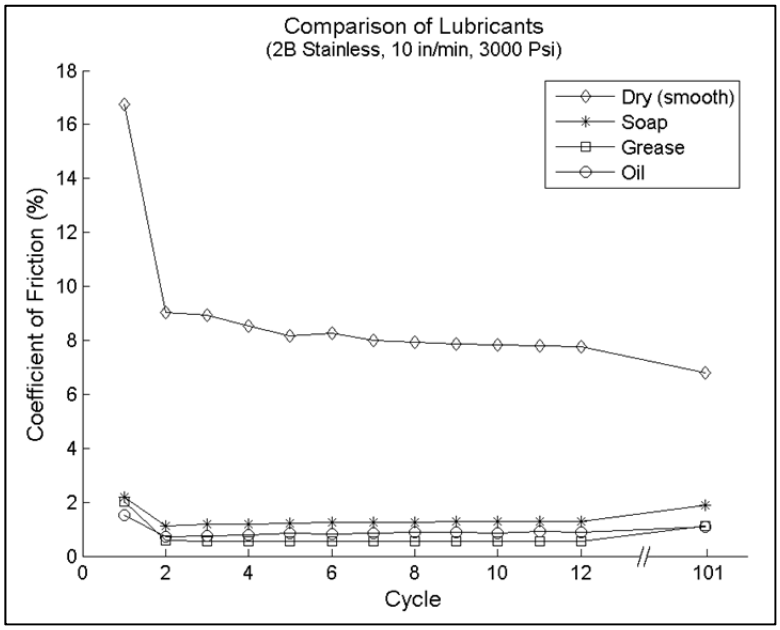


Constant parameters: 2B, 10 in/min, 3000 psi

Figure 6.5.2-2: Comparison of CoF for Different Lubricants

The values for CoF for the soap, oil, and grease were all lower when compared to the 1500 psi tests. Again this shows that as the pressure increases, the CoF decreases. Oil, grease, and soap all performed very similar with this group of tests. The grease and soap tests started out at just over 2%, and the oil performed the best for the first cycle producing a CoF of 1.5%.

This next group of tests that were examined was for the 4500 psi load level. Each of these tests used the No. 2B stainless steel, a sliding speed of 10 in/min, and 4500 psi load. Figure 6.5.2-3 plots the CoF for this group. Once again the grease, oil and soap are very low compared to the dry test.



Constant parameters: 2B, 10 in/min, 4500 psi

Figure 6.5.2-3: Comparison of CoF for Different Lubricants

This plot also includes a graphite test demonstrating the poor performance. The second graphite test used less graphite and a smooth PTFE pad, which resulted in better performance. The PTFE pads did not deform much in comparison to the test that used more graphite and dimpled PTFE. Also, as is illustrated in Figure 6.5.2-4, the CoF values were quite a bit lower for the test that used the smooth PTFE pads and light coating of graphite (Test 2B,Gph,2.5,4500). The friction was lower for the beginning cycles, and then they both eventually ended up converging to the same point towards the end of the cycles. Note that the 2 tests were also done at different speeds, which also had an effect in the 2.5 in/min test having lower values. Based on the performance of these 2 tests it was decided that graphite is not an acceptable lubricant, therefore subsequent test were not run.

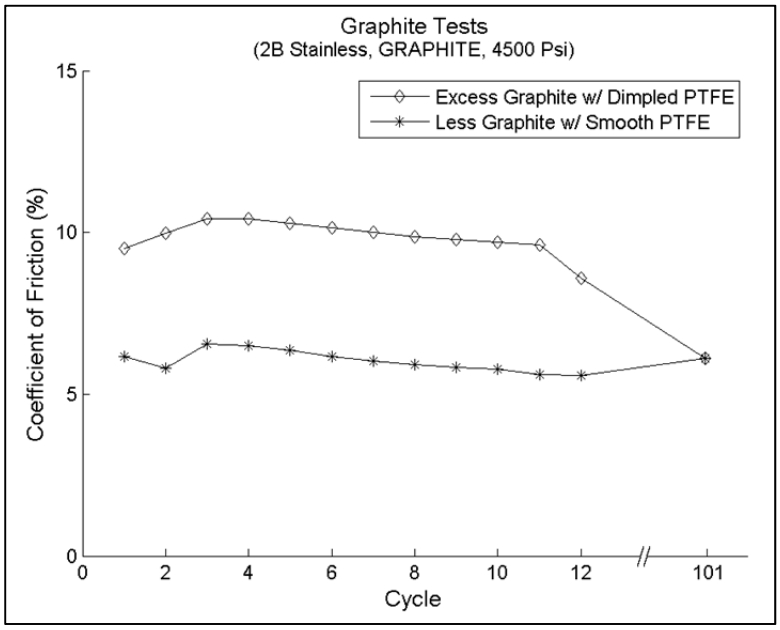


Figure 6.5.2-4 Results of Graphite Lubricant

In summary the oil lubricant tests performed very well. The oil kept the pads lubricated throughout the testing. The CoF values dropped significantly using oil versus dry. The grease tests performed just as well as the oil tests in most cases and since it repels water is assumed that it won't wash away as easily as the oil may during a potential rain storm. The soap tests also performed well. The friction values for the soap were often not much higher than what was seen for the oil and grease tests. The graphite performed poorly, therefore it is not recommended for use in lateral bridge slides. It is also recommended that dry PTFE not be used with No. 2B stainless steel, since these tests also gave much higher CoF values than what can be obtained using lubricants. Non-lubricated PTFE could still be used, however No. 8 mirror finish Stainless steel would be recommended. The CoF values listed in the *AASHTO LRFD Bridge Design Specifications* could be used for the design.

6.5.3 Surface Roughness

The 3 types of surface roughness that were tested include No. 2B stainless steel, No. 1 stainless steel, and carbon steel. Overall, the 2B stainless steel produced much lower CoF values than the No 1 RR stainless as well as the carbon steel, therefore #2B stainless steel is recommended for bridge slides due to its lower costs when compared to No. 8 mirror finish stainless steel and adequate performance.

6.5.4 Sliding Speed

The sliding speed of 2.5 in/min produced CoF values slightly lower than the 10 in/min speed, however the team felt that the 2.5 in/min speed was unrealistically slow for a lateral bridge slides. A travel speed of 10 in/min is recommended for project specifications.

6.6 Recommended Guidelines

The research team reviewed the testing data and synthesized the results into proposed provisions for the guidelines. Several parameters were left out of the recommendations due to poor performance when compared to the others.

- RR stainless Steel
- Carbon Steel
- No lubricant
- Graphite Lubricant
- Dynamic Friction

The parameters that are recommended for the guideline are:

- #2B finish stainless steel
- Contact Pressure
- Lubricants (Grease, Oil)
- Travel speed limited to less than 10 in./min.

The project panel in conjunction with the project team decided to not include soap lubricants in the guidelines. The reasoning was that there are different liquid soaps in the market and no national specifications for soap. This would complicate specification development and potentially lead to proprietary product issues. Another reason for not including soap is the fact that the grease and oil lubricants perform very well, are readily available, and are cost effective.

Rough surface finish steels do not provide the desired low friction values. If designers or contractors choose to use non-lubricated pads, the use of mirror finish stainless steel would be recommended. The *AASHTO LRFD Bridge Design Specifications* includes provisions for this type of design, therefore it is recommended for use.

The #2B finish stainless steel performed very well in all tests, therefore it is recommended to use in lateral bridge slides. It is roughly 50% less expensive to use #2B stainless than the #8 mirror stainless. Since the #2B stainless steel performed similar to the #8 mirror stainless steel, it is recommended that either one of these stainless finishes be used for SIBC. The benefit of this material when compared to the cost is the basis for this recommendation.

The approach for the guide specification was to produce a table that is similar to Table 14.7.2.5-1 of the *AASHTO LRFD Bridge Design Specifications*. The parameters in this table are:

- Contact pressure
- Temperature
- PTFE type
- Lubrication

The temperature parameter was not varied in this research because bridge moves are typically completed during normal temperature ranges. If a contactor desired to slide a bridge during extreme temperatures, the ratios of the values of the AASHTO table could be used to adjust the recommended values in the guidelines.

The lubrication values in the AASHTO table are based on the use of Silicone Grease. The guideline table should also include oil lubricants. These lubricants would not be recommended for long-term performance; however they are acceptable for a short-term operation such as a bridge slide.

The AASHTO table covers different types of PTFE. Filled PTFE is a good material for bridge bearings. The fillers improve the long-term performance of a bridge bearing. The short-term nature of a bridge slide makes the use of unfilled PTFE acceptable. The testing focused on smooth and dimpled unfilled PTFE, because it is an ideal material for short-term operations where lower friction is more desirable than long-term durability.

Dynamic friction values were discarded because in all cases, the initial cycle 1 static friction was greater than the dynamic friction of subsequent re-start friction. This initial start-up static friction will occur in all cases; therefore the slide mechanism should be designed to overcome the force generated by this friction.

The results of the testing were reduced down to the parameters described above. Table 6.6-1 shows the results. The values for mirror finish stainless steel noted in Table 14.7.2.5-1 of the *AASHTO LRFD Bridge Design Specifications* are also included for comparison.

Coefficient of Friction (%) (Static Friction from Cycle 1)							
Pressure (psi)	**500**	**1000**	**1500**	**2000**	**3000**	**4500**	**8000**
AASHTO (stainless mirror finish)	4.00	3.00		2.50	2.00	2.00	2.00
Soap (stainless 2B finish)			5.66		2.18	2.45	1.44
Grease (stainless 2B finish)			5.15		2.02	1.06	0.67
Oil (stainless 2B finish)			4.04		1.51	2.75	0.90
Note: Dimpled, Lubricated, Unfilled, virgin PTFE pads were used							

Table 6.6-1 Data Reduction Table

The results were plotted to compare the contact pressure with the CoF. Figure 6.6-2 shows the results of these plots.

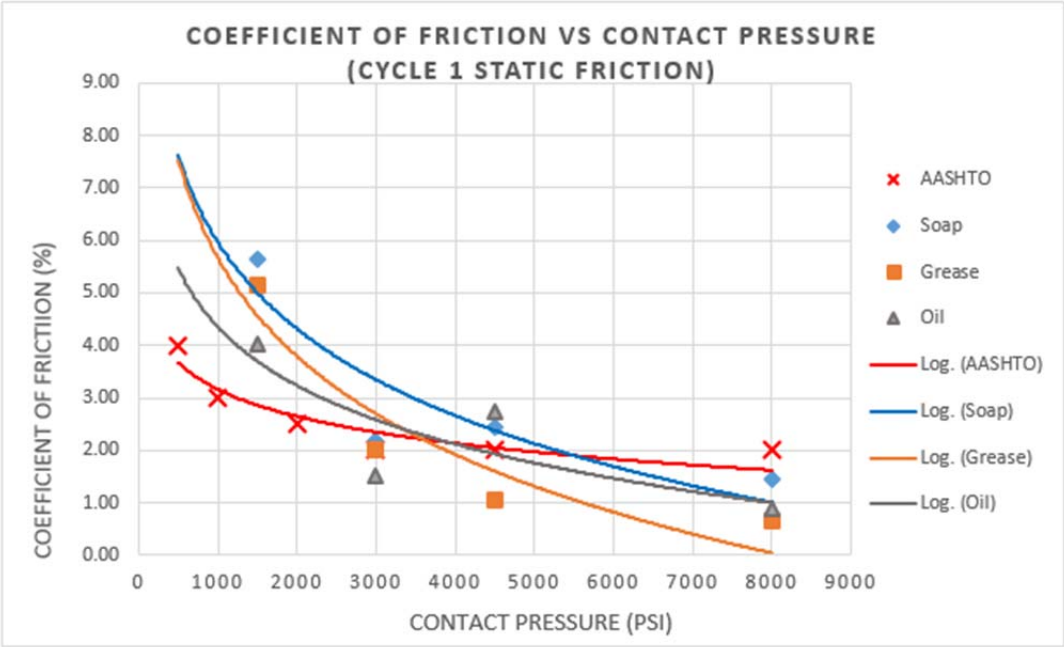


Figure 6.6-2 Comparison of Recommended Specification Data

The AASHTO values plot on a relatively smooth curve. The minor variations are most likely due to rounding of values. There was some variability noted in the testing results. This variability is normal when a limited number of tests are performed. The AASHTO data follows a logarithmic curve; therefore a logarithmic curve was fit to the testing data. As expected, some of the data falls below the curves and some falls above. The guide specification recommendations should encompass all or nearly all the data to ensure conservatism in practice. The recommendation is to increase the curve values by 20 percent and round the results to the nearest 0.5%. Figure 6.6-3 shows the results of this adjustment. This table represents the recommended values to be used in the guide specification.

Recommended Design Values for Coefficient of Friction (%)					
Pressure (Psi)	**500**	**1000**	**2000**	**3000**	**≥ 4500**
Dielectric Grease Lubricant (SAE-AS8660)	9.5	7.0	5.0	3.5	3.0
Motor Oil Lubricant (10W-40 viscosity)	7.0	5.0	4.0	3.5	3.0
Note: Values are for dimpled and lubricated unfilled virgin PTFE at approximately 50 degrees Fahrenheit					

Table 6.6-3 Recommended Guideline Design Table

A decision was made by the project team and research panel to exclude soap from the recommended guideline table. This was due to the fact that the grease and oil performed equally as well and specifications for dish soap essentially do not exist. The guideline commentary will suggest allowing soap as a contractor alternate based on experience.

CHAPTER 7

Guideline Development

7.1 Approach

Chapter 4 details the research and recommended provisions for development of a guideline for tolerances for PBES. Chapter 5 details the research and recommended provisions for development of a guideline for dynamic effects for bridge systems.

The approach for development of the guidelines was to take the research results and develop guidelines and commentary that can be used by bridge practitioners in the design and construction of ABC projects. These guidelines are not stand-alone documents. They fill particular facets of the design and construction process for ABC projects.

The 2 completed guidelines are included in this report. Appendix C contains the *Guidelines for Prefabricated Bridge Elements and Systems*. Appendix D contains the *Guidelines for Dynamic Effects for Bridge Systems*.

7.2 Guideline Section and Article Development

The team was charged with developing a guideline in AASHTO format. This is not a stand-alone design specification, but a supplement the *AASHTO LRFD Bridge Construction Specifications* and the *AASHTO Guide Specifications for Accelerated Bridge Construction*. The key features of this guideline include:

1. Two column format: Specifications in the left column, commentary in the right column.
2. A "section" is akin to a chapter in other documents.
3. Articles are numbered using number headings (1.1, 1.1.1, 1.1.1.1, etc.)
4. Commentary headings start with a Capital C.
5. Tables of contents are generated for each section as opposed to an overall document table of contents.
6. The first article in each section contains the scope of that section.
7. Notations for each section are included in the front of each section. Reference to the applicable articles is also identified.
8. References to current AASHTO provisions do not include the actual provision number, just the provision title. This is done to address possible re-organization of AASHTO documents.

A guideline should not be treated similar to a full specification. Instead, it is a recommended practice that may be followed. AASHTO "specifications" make use of the terms "shall", "should", "may", and "recommended". The following describes how these terms are interpreted:

- The term "shall" denotes a requirement for compliance with the specifications.
- The term "should" indicates a strong preference for a given criterion.

- The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.
- The term "recommended" is used to give guidance based on past experiences.

The recommended guideline document makes use of the words "should", "may" and "recommended" in most provisions, since it is a guideline and not a specification.

The following sections contain information on the development of each specification section.

7.3 Guidelines for Prefabricated Bridge Elements and Systems Tolerances

7.3.1 Section 1: Introduction

The provisions in this section serve as an introductory section of the overall guideline. Section 1 includes definitions of common prefabricated elements and systems and brief description of different types of bridge systems.

7.3.1.1 Definitions

The definitions were vetted through the project panel and the AASHTO T-4 Construction Technical Committee. These definitions are also used in national project databases. Use of common definitions aids in the management and dissemination of information across the country. The AASHTO T-4 Technical Committee requested that the team not change these definitions for this report and the proposed guidelines. The definitions are organized by major category for ease of use.

7.3.1.2 Management of Tolerances

This section contains recommended approaches to management of tolerances. The key variation from current practice is that the designer should be responsible for specifying tolerances within the contract documents. This is due to the fact that tolerances will affect the width of joints between elements, the materials used in the joints, and the layout of the structure.

7.3.2 Section 2: Fabrication Tolerances

This section includes the recommended fabrication tolerances for common prefabricated elements and systems. The basis of this section is described in Chapter 4 of this report. This section contains details and guideline tolerances that can be used in project contract documents.

7.3.3 Section 3: Erection Tolerances

This section includes the recommended erection tolerances for common prefabricated elements and systems. The basis of this section is described in Chapter 4 of this report.

This section describes the importance of structure layout using working points and working lines. Two types of horizontal layout methods are proposed. The first is the "Center of Element Method". This method is based on erecting elements by placing them at a distance specified from the working line to the

center of the element. The goal being to have the element placed in a horizontal position where the location of the element within the structural frame is critical for the performance of the bridge. This is the most common form of element layout and is appropriate for most elements. The second method of layout is "Surface of Element Method". This method is based on erecting elements by placing them at a distance specified from the working line to one face of the element. The goal being to have the element placed in a horizontal position where the faces of adjacent elements line up to produce an aesthetic structure. The most common use of this method would be for faces of retaining walls or abutments.

For certain structures, both methods of horizontal layout can be used. For example, a retaining wall could use center of element layout for the longitudinal spacing of the wall panels, and face of element layout for the transverse layout of the panels. The result would be a wall that has reasonable joints between panels and a smooth face. If face of element methods is used for layout, the same basis should be used for the fabrication of the element. For example, the location of inserts in a wall panel would be measured from the specified element face, not the centerline of the element.

7.3.3.1 Accumulation of Tolerances

The fabrication and erection tolerances of elements are based on the assumption that joints will be used between the elements to accommodate the specified tolerances. Subsequent sections of the guideline outline the role that fabrication and erection tolerances play in the development of a joint width and joint width tolerance. In general, larger tolerances require larger joints to accommodate them.

In some instances, joints are not used between elements during erection. This is the case for items that are dry stacked such facing block on some mechanically stabilized earth retaining walls and precast longitudinal barriers. If joints are not specified, there is the potential for the accumulation of tolerances. For example, if 10 precast panels that are one foot thick are stacked on top of each other, the resulting structure will inevitably be more or less than 10 feet. The tolerances of the elements will lead to mis-fits and minor gaps between the elements. In most cases the resulting structure will be taller or longer than anticipated. Provisions were included in the guidelines for the management of stacked elements. In general, the designer needs to accommodate the accumulation of tolerances through the use of closure pours or variable overall dimensions.

7.3.3.2 Bridge System Tolerances

Tolerances for bridge systems are included in this section. The basis for these tolerances is described in Section 3.1.1 of this report. In general, they are based on the experience of the Utah DOT, an agency that has moved more bridges than any other in the United States. Tolerances for bridge systems are typically accommodated by closure joints at the ends of bridges.

7.3.4 Section 4: Joints and Connections

This section contains the results of the Monte Carlo simulations described in Chapter 4 of this report. Guidelines for common connection details used in the United States are shown. The basis for the selection of the details is twofold:

1. The PCI Northeast Bridge Technical Committee: This committee has published documents showing recommended details for connections between common precast elements.
2. NCHRP Project 12-102: A portion of this project included a questionnaire to all 50 states that asked for information on what forms of ABC are in use. The results of that questionnaire formed a basis for some of the details chosen.

The general list of joints and connections covered include:

- Vertical Joints: Joints between side-by-side elements
  - Approach Slab to Approach Slab
  - Approach Slab to Sleeper Slab
  - Sleeper Slab to Sleeper Slab
  - Pier Cap to Pier Cap
  - Panel to Panel
  - Footing to Footing
  - Wall Cap to Wall Cap
  - Full-Depth Deck Panel to Full-Depth Deck Panel
- Horizontal Joints: Joints between stacked element
  - Column to Footing
  - Column to Pier Cap
  - Column to Prestressed Pier Cap
  - Wall Cap to Wall Panel
  - Approach Slab to Backwall or Sleeper Slab Seat
  - Wall Panel to Footing
  - Column to Column
  - Wall Panel to Wall Panel
- Connections
  - Grouted Splice Couplers
  - Pockets
  - Reinforced Pockets
  - Integral Abutment Pile Pockets
  - Reinforced Closure Joints

Schematic details were developed for each joint and connection depicting the applicable tolerance that affects the joint widths. From these details, equations were written for the tolerances that could affect the joint tolerance. A certain degree of logic was applied in order to keep the results reasonable. For example, it was assumed that the edge squareness and edge flatness do not occur at the same location, therefore they are not additive. This is a logical assumption because the squareness affects the joint at the corners, but the flatness affects the joint along its length.

7.3.4.1 Specifying Joints and Joint Tolerances

This section also contains information on how to specify joints and joint tolerances. The controls on these specifications include:

1. The minimum allowable joint width: This is normally based on the material that is to be placed within the joint, and the method for placement of the material. For example, it may be possible to place a flowable non-shrink grout in a joint that is as narrow as ½"; however, a joint may need to be 1½" wide to allow for placement of concrete.
2. The joint width tolerance: This is a function of the specified element fabrication tolerances. Larger tolerances results in larger joints.

Example of a specified joint width would be:

Specified vertical joint width (side-by-side elements):

$$S_{jw} = t_{min\_jw} + T_{jw} \qquad \text{(Guideline Equation 4.5.1-1)}$$

Specified horizontal joint thickness (stacked elements):

$$S_{jt} = t_{min\_jt} + T_{jt} \qquad \text{(Guideline Equation 4.5.1-2)}$$

where:

S_{jt}	=	specified joint thickness (in.)
S_{jw}	=	specified joint width (in.)
t_{min_jt}	=	minimum tolerable joint thickness (in.)
t_{min_jw}	=	minimum tolerable joint width (in.)
T_{jw}	=	joint width tolerance (in.)
T_{jt}	=	joint thickness tolerance (in.)

The detail call out for a particular joint would be:

Horizontal joint detail call out = $S_{jt} \pm T_{jt}$
Vertical joint detail call out = $S_{jw} \pm T_{jw}$

A review of these equations indicates that if the maximum negative joint tolerance was present, the result would be the minimum allowable joint width. If the maximum positive joint tolerance was present, the results would be the minimum allowable joint width plus 2 times the joint tolerance.

7.4 Guidelines for Dynamic Effects for Bridge Systems

7.4.1 Section 1: Introduction

The provisions in this section serve as an introductory section of the overall guideline. Section 1 includes definitions of common prefabricated systems and system features.

7.4.1.1 Definitions

The definitions were vetted through the project panel and the AASHTO T-4 Construction Technical Committee. These definitions are also used in national project databases. Use of common definitions aids in the management and dissemination of information across the country. The AASHTO T-4 Technical Committee requested that the team not change these definitions for this report and the proposed guidelines.

7.4.1.2 Bridge System Types

This article includes descriptions of the most common forms of bridge systems in use in the United States. It also makes note of other specialized bridge system methods such as barge installation and longitudinal launching. These methods are specialized on a site by site basis. They require careful coordination between the contractor and the engineer.

7.4.2 Section 2: Dynamics of Bridge Systems

This section contains the results of 2 of the studies that were completed under this project. The SPMT portions of the dynamic studies are reported in Chapter 5 and the lateral slide portion of the dynamic studies is reported in Chapter 6.

7.4.2.1 Self-Propelled Modular Transporter Systems

As described in Chapter 5, the approach for dynamics of SPMT systems is akin to seismic analysis. In order to keep this analysis reasonable, the Uniform Load Method of the AASHTO LRFD Bridge Design Specifications was chosen for the guideline.

General Approach to Dynamic Analysis

The general approach to the dynamic analysis is as follows:

1. Apply a unit load in the direction being studied
2. Calculate the maximum displacement of the system under the unit load
3. Calculate the percent of capacity of the SPMTs
4. Calculate the stiffness of the system under the unit load (load/displacement)
5. Calculate the period of the system
6. Determine the elastic dynamic response coefficient
7. Determine the equivalent dynamic load applied to the system
8. Design the falsework and bridge for the dynamic forces combined with non-dynamic forces

In order to complete a dynamic analysis of a bridge supported on a SPMT, the user of this guideline needs to know the stiffness of the falsework. In most cases, the bridge designer does not know the design of the falsework during design development. To resolve this issue, a simplified design equation was developed that does not require the stiffness of the falsework. This is based on the 2 design response spectra that were developed (horizontal and vertical). Figure 7.4.2.1-1 and 2 show the 2 response spectra plots.

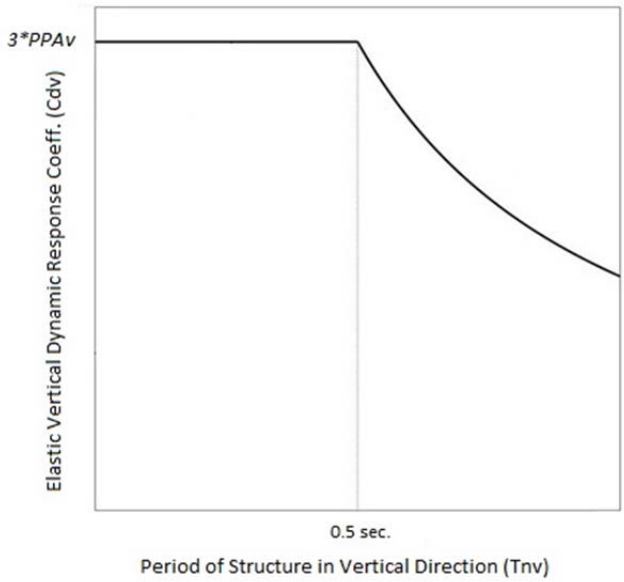


Figure 7.4.2.1-1 Vertical Design Response Spectrum

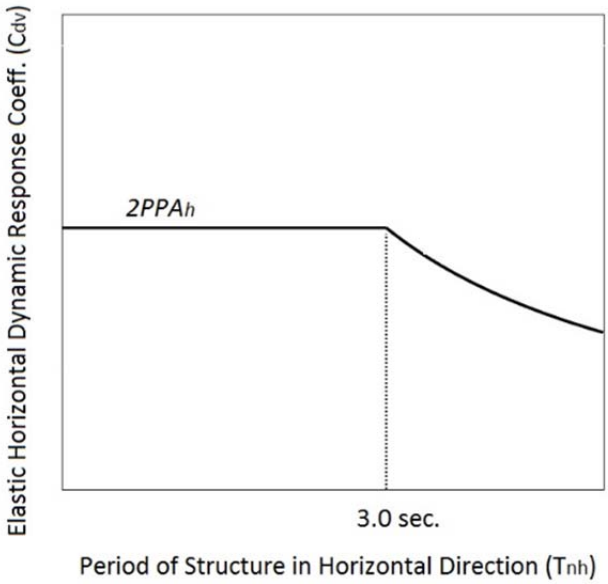


Figure 7.4.2.1-2 Horizontal Design Response Spectrum

A review of these 2 plots indicates a plateau on both plots for short periods. In many cases, the period of the final system will be below the thresholds where the dynamic response coefficient decreases; therefore the maximum coefficient will be used. In the case where the designer does not know the

stiffness of the falsework, the response coefficient can be conservatively calculated based on the maximum value. Simple equations were developed where a design can calculate the equivalent static load. These equations are included in Articles 2.4.2.1 and 2.4.3.1.

Vertical Dynamics

The research team developed a response spectrum approach that is very similar to the "Uniform Load Method" of seismic analysis that is included in the AASHTO LRFD Bridge Design Specifications.

In this method, the structure is assumed to act as a single mass-spring oscillator. The period of the structure is calculated based on its response to a uniform load applied vertically to the system. In most cases, the falsework and SPMT can be considered to be rigid bodies, thereby limiting the response to the bridge structure.

The method not only accounts for the stiffness of the bridge system, but also the magnitude of the load on the SPMTs. Testing showed that the magnitude of the dynamic effects varied significantly with the load on the SPMT; therefore it is covered in the specification.

During testing, the magnitudes of vertical accelerations were found to be quite high, which was expected since the SPMT was run through rough terrain at maximum speeds, representing an ultimate load. It is necessary to use vertical dynamics to perform service limit state checks. Subsequent sections of the guideline will cover the approach used for this.

Horizontal Dynamics

The research team again developed a response spectrum approach that is similar to the vertical dynamic approach. In this method, the structure and falsework is assumed to act as a single mass-spring oscillator. The period of the system is calculated based on its response to a uniform load applied horizontally to the system. In most cases, the bridge structure can be considered to be a rigid body, thereby limiting the response to the falsework.

As with the vertical dynamics, the method accounts for the magnitude of the load on the SPMTs. Testing showed that the magnitude of the dynamic effects varied significantly with the load on the SPMT; therefore it is covered in the specification.

During testing, the magnitudes of horizontal accelerations and decelerations were found to be very high, which was expected since the SPMT was stopped abruptly while running at maximum speeds. This represents an emergency stop condition, which is considered to be an ultimate load case. This situation would only occur in very rare situation (maximum braking applied at high speed). The research team recommends that this be treated similar to an extreme event.

The recommendation is to reduce these extreme accelerations in a similar manner as in the AASHTO LRFD Bridge Design Specifications through the use of response modification factors. The concept is that minor damage such as yielding or minor sliding of falsework can be accepted in response to an emergency stop situation. Ductile falsework systems would be allowed to yield, resulting in lower forces. Stiffer falsework systems would be designed for higher forces. The values chosen for response modification factors were taken from the *AASHTO Guide Design Specifications for Bridge Temporary Works*, which contains provisions for seismic design of falsework. The values chosen for R factors were as follows:

- Limited Ductility: Braced Frames and Stiff Containers = 2.0
- Medium Ductility: Towers braced with yielding cables and chains = 2.5.

7.4.2.2 Load Combinations

The testing performed on the SPMT was meant to be an "ultimate load condition. The machine was run at speeds and over terrain that would most likely never exist in a typical bridge move. For instance, the "long run" case involved driving the SPMT at full speed over dirt terrain with ruts that were over 9" deep. Starting and stopping was also pushed to the limits of the machine. The tires on the SPMT actually locked in a few situations when the machine was lightly loaded.

Service Load Combination Check for the Bridge

The bridge designer needs to verify that the bridge can be moved without undue damage. Several checks are recommended to ensure this. The first is the Service 1 Load combination. It is recommended that this load combination be used for:

- Cracking in concrete decks
- Cracking in Prestressed concrete beams
- Bridge falsework vertical capacity (if designed using working stress methods)
- Working loads on the SPMTs

The load factor for vertical dynamic loads is set at 0.4. This value was calculated by reducing the travel speed from 4 mile per hour (test velocity) to 2.5 miles per hour, which is the recommended specification maximum travel speed. The loads were found to be a function of the speed squared ($2.5^2/4^2 = 0.39$, rounded to 0.4). See Chapter 3 of this report for more information on this approach.

Strength Load Combination for the Bridge

It is recommended that the strength load combination be used for the Flexural Capacity of the bridge under vertical dynamic loads. In this load combination, the dead loads are factored according to the *AASHTO LRFD Bridge Design Specifications* and the vertical dynamic load is based on the recommended guidelines. A load factor of 1.0 is used for the vertical dynamic loads since the testing was performed to mimic the ultimate load that the system would experience during the move.

Load Combinations for Falsework and SPMTs

The load combination for the design of the falsework and SPMTs is based on the *AASHTO Guide Design Specifications for Temporary Works (2017)* with a minor modification. A service load combination us recommended because most temporary works and SPMT designs are completed using service loads. The minor modification is for the horizontal dynamic loads. The design is similar to a seismic event, but not the same, since the probability of occurrence is much higher than a seismic event.

Load Combination ASD12 of the *AASHTO Guide Design Specifications for Temporary Works (2017)* accounts for seismic forces. A load factor of 0.53 is specified for these loads. In order to account for the increased probability of the maximum horizontal dynamic loads, a load factor of 0.75 is recommended. This is the same load factor that is applied to live load on falsework, which should be appropriate for dynamic loads. Other loads and load factors specified in Load Combination ASD12 should be applied to the loads specified in this provision.

7.4.2.3 Lateral Slide Systems

This section describes the need for dynamic analysis of sliding forces for lateral slide installations. It notes that vertical and horizontal dynamics are not critical due to the very low velocities used and the

stability of the slide mechanisms. This leads to a quasi-static event. The dynamic factors that do need to be accounted for are the friction in the slide system. Chapter 6 of this report includes detail on the research performed on sliding friction including the materials and lubricants chosen.

Materials for sliding systems are specified based on common practice in the industry. PTFE is the preferred material for sliding surfaces; however there are several types of PTFE in the market. The team has recommended the use of unfilled PTFE due to is lower friction values. Filled PTFE is necessary for long-term applications such as bridge bearings. The most significant change from current practice is to allow the use of No. 2B finish on stainless steel sliding plates. This material is approximately on half the cost of No. 8 "mirror" finish stainless steel (which is used in bearings). No. 2B finish stainless steel has higher friction values than No. 8 finish stainless steel, however the increases are marginal.

Lubricants are not often used in bridge bearings due to the need to re-apply them over time, which is an undesirable maintenance task. It is reasonable to use lubricants on lateral slide bridge moves, since it is a short-term operation. Three types of lubricants are recommended in the guideline based on performance in the testing.

Steel sliding materials and lubricants that should not be used are noted in the commentary. There are steel on steel (combined with grease) proprietary systems that are in use in the market. These systems can be allowed and design based on the manufacturer's knowledge of the friction in the system.

The results of the friction testing are listed in a design table that is similar to the friction tables for bearings in the AASHTO LRFD Bridge Design Specifications. The research confirmed that the CoF reduce with increased load on the sliding bearing.

7.4.2.4 Example Calculations

Appendix A of this section includes 2 sets of example calculations for dynamics of an SPMT system. The example covers both horizontal and vertical dynamics. Flow charts were developed to clearly demonstrate the design process.

CHAPTER 8

Implementation Plan

8.1 Goals for Implementation

The title of this research project is "Recommended Guidelines for PBES Tolerances and Dynamic Effects". The goal of the work is to develop an AASHTO format guideline that ultimately will be adopted by AASHTO. NCHRP has provided the tools for success in this effort. Project panel members include members of the AASHTO Subcommittee on Bridges and Structures (SCOBS) as well as agency bridge engineers. The panel has worked with the project team to ensure that the knowledge gaps in tolerances and dynamics are addressed with this document.

The main goals for implementation include:

- Develop an AASHTO Format Document
- Work with the AASHTO T-4 Construction Technical Committee for adoption of the Document by AASHTO SCOBS
- Provide training opportunities for practicing engineers to improve use of the document

8.2 Proposed Implementation Plan

The recommended goals for implementation of this research are as follows:

1. Develop an AASHTO Format Document: This goal was built into the development of the specification. The project team paid strict attention to developing a guide specification that was according to AASHTO standards including format and content. In the opinion of the project team, this goal has been achieved.
2. Adoption of the Document by AASHTO SCOBS: This goal requires significant effort in distributing the document to all AASHTO SCOBS members, and disseminating comments from the voting members. The project team plans to work with the AASHTO T-4 Technical Committee for Construction (the committee that has been assigned the management of ABC at AASHTO) to implement this guide. These guidelines are interconnected with the ABC Guide Specification that was recently adopted by AASHTO. There have been discussions with the T-4 Committee about potentially merging these guidelines into the ABC Guide Specifications. This will be a significant effort, but ultimately worth the time.
3. Provide training opportunities for practicing engineers to improve use of the document: The best mechanism for training at this time is the Florida International University ABC University Transportation Center (FIU ABC-UTC). The mission of this center is as follows:

   The ABC-UTC has assembled an experienced, knowledgeable, and engaged group of bridge academics and engineers who collectively will provide the transportation industry with the tools needed to effectively and economically utilize the principles of ABC to enhance mobility, and safety and produce safe, environmentally friendly, long-lasting bridges.

The project team has already engaged the ABC-UTC for training in the use of these guidelines. Michael Culmo has already presented 2 national webinars on this project (with permission from NCHRP): one for tolerances and one for dynamics.

The center also holds a biennial ABC Conference that includes half-day workshops. Michael Culmo will be giving a presentation during a workshop on SPMTs for the 2017 Biennial Conference in December.

FHWA maintains training programs under the National Highway Institute. The development of a training module for ABC would be beneficial. Other opportunities might include web based modules for training, which could be developed through the State Transportation Innovation Councils. Through this program, a state can work with FHWA to develop deployment of this innovation.

References

AASHTO LRFD Bridge Design Specifications, Seventh Edition, American Association of State Highway and Transportation Official, Washington, DC. (2014)

AASHTO/AWS, Bridge Welding Code, AASHTO/AWS D1.5, American Associates of State Highway and Transportation Officials, Washington, D.C. and the American Welding Society, Miami, FL (2010).

AASHTO/NSBA, Steel Bridge Erection Guide Specification, National Steel Bridge Alliance, Chicago, IL (S10.1-2014)

AASHTO/NSBA Steel Bridge Fabrication Guide Specification, National Steel Bridge Alliance, Chicago, IL (S2.1-2008)

AASHTO/NSBA Steel Bridge Fabrication QC/QA Guide Specification, National Steel Bridge Alliance, Chicago, IL (S4.1-2002)

ASME, Dimensioning and Tolerancing, Engineering Drawing and Related Documentation Practices, ASME Y14.5-2009, The American Society of Mechanical Engineers

Aktan, H., Et al, Accelerated Bridge Construction and Structural Move - Workshop, Western Michigan University, Kalamazoo, MI, pp 50, (2014)

Ardani, A., Et al, One-Weekend Job, Rapid Removal and Replacement of 4500 South Bridge in Salt Lake City, Utah, Transportation Research Record, *Journal of the Transportation Research Board*, No. 2200, Transportation Research Board of the National Academies, Washington, D.C., pp 12-16, (2010)

Beerbower, R.E., Tolerances, *Proceedings*, Panel on Plant Production, Precast Prestressed Concrete Institute, Chicago, IL. (1964)

Campbell, T.I. and Manning, D.G., Coefficient of Friction in the TFE Slide Surface of a Bridge Bearing, Queen's University, Ontario Ministry of Transportation, Ontario Canada (1990)

Culmo, M., Accelerated Bridge Construction - Experience in Design, Fabrication, and Erection of Prefabricated Bridge Elements and Systems, FHWA Publication No. HIF-12-013, Federal Highway Administration, Washington, D.C., pp 347, (2011)

Dolce, M., Et al, Frictional Behavior of Steel-PTFE Interfaces for Seismic Isolation, *Bulletin of Earthquake Engineering*, 3(1), pp. 75-99, (2005).

FHWA, Slide-In Bridge Construction Implementation Guide, Project: F-ST99 (232), U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., pp 168, (2013)

Gleich, H.A., Et al., Joints in Precast Parking Structures, *PCI Journal*, September-October, 2007, Precast Prestressed Concrete Institute, Chicago, IL.

Goodspeed, C., Transverse Joint Configuration Development and Testing for a Modular Bridge Deck Replacement, University of New Hampshire, (2011)

Hanskat, C.S., Et al., Specification for Tolerances for Precast Concrete, ACI ITG-7-09, American Concrete Institute, Farmington Hills, MI, (2009)

Innovative Bridge Designs for Rapid Renewal: ABC Toolkit, SHRP2 Report S2-R04-RR-2, Strategic Highway Research Program 2, Transportation Research Board, Washington D.C., pp 323. (2013b)

Issa, M.A., Et al, Design Considerations of Full-Depth Precast/Prestressed Concrete Bridge Deck Panels, *Proceedings*, Concrete Bridge Conference, PCI, Nashville, Tennessee, (2002)

Kim, S., Et al, Analytical Assessment of the Effect of Vertical Earthquake Motion on RC Bridge Piers, American Society of Civil Engineers (ASCE), Vol. 137, pp 252-260, (2011)

Lanier, M., Et al., Tolerances for Precast and Prestressed Concrete, PCI Committee on Tolerances, Committee Report, *PCI Journal*, January-February, Precast Prestressed Concrete Institute, Chicago, IL, (1985)

LaViolette, M., Et al, Bridge Construction Practices Using Incremental Launching, Iowa State University (ISU), Ames, Iowa, pp 106, (2007)

PCI, Design and Typical Details of Connections for Precast and Prestressed Concrete, Precast Prestressed Concrete Institute, Chicago, IL, (1988)

PCI, Envelope Tolerances for Architectural Precast - Designer Note-24, Precast Prestressed Concrete Institute, Chicago, IL.

PCI, Guide Specification for Architectural Precast, Precast Prestressed Concrete Institute, Chicago, IL, (2010)

PCI, Tolerance Manual for Precast and Prestressed Concrete Construction - MNL-135-00, Precast Prestressed Concrete Institute, Chicago, IL, (2000)

Ralls, M.L., Accelerated Bridge Construction Workshop, *Workshop Report*, Iowa Department of Transportation, Ames, IA, pp 87, (2008)

Ralls, M.L., Manual on use of Self-Propelled Modular Transporters to Remove and Replace Bridges, *Publication FHWA-HIF-07-022*, Federal Highway Administration, American Association of State Highway and Transportation Officials, National Cooperative Highway Research Program, and Florida Department of Transportation, pp 118, (2007)

Rosvall, E. and Halling, M., Data Analysis of Utah's I-80 Bridges - For the Use in the Development of Accelerated Bridge Construction Standards, Utah State University, Logan, Utah, pp 90, (2010)

Rosvall, E. Et al, Induced Stresses from Lifting and Moving Highway Bridges with Self-Propelled Modular Transporters, Utah State University, Logan, Utah, pp 17-25. (2010)

Schneider, J.A. and Et al., Product Tolerances for Precast and Prestressed Concrete, PCI Committee on Tolerances, *Committee Report*, PCI Journal, January-February, 1993, Precast Prestressed Concrete Institute, Chicago, IL.

Seraderian, R.; Culmo, M.P.; Et al, Guidelines for Accelerated Bridge Construction using Precast/Prestressed Concrete Elements including Guideline Details, Precast Prestressed Concrete Institute - Northeast, Belmont, MA.

SHRP2, ABC Standard Concepts: The Lateral Slide -Addendum to Innovative Bridge Designs for Rapid Renewal, ABC Toolkit, SHRP 2 Renewal Project R04, Publication Draft, Strategic Highway Research Program 2, Transportation Research Board, Washington D.C., pp 70. (2013)

Tang, B. and Ralls, M.L., Prefabricated Bridge Elements and Systems in Japan and Europe, Federal Highway Administration, Office of International Programs and the American Association of State Highway and Transportation Officials, pp xii, 13, (2005)

Taylor, J.C. and Stanton, J.F., Friction Coefficients for Stainless Steel (PTFE) Teflon Bearings, University of Washington, Seattle WA, Wisconsin Department of Transportation, Madison WI, Federal Highway Administration, Washington DC. (2010)

Tipping, E.G., Et al, Specification for Tolerances for Concrete Construction and Materials (ACI 117-10) and Commentary, American Concrete Institute, Farmington Hills, MI, (2010)

Utah DOT, Accelerated Bridge Construction SPMT Process Manual and Design Guide, Utah Department of Transportation, Salt Lake City, Utah, pp 84. (2009)

Utah DOT, Structures Design and Detailing Manual, Utah Department of Transportation, Salt Lake City, Utah (2015)

Vander Wal, M. and Walker, H.C., Tolerances for Precast Structures, PCI Journal, July-August 1976, Precast Prestressed Concrete Institute, Chicago, IL.

White, Et al., Crash Testing of Various Textured Barriers, *Report No. FHWA/CA/TL-2002/03*, California Department of Transportation, Sacramento, CA, (2002)

Wipf, T.J.; Phares, B.M.; Abendroth, R.A.; Wood, D.L.; Chang, B., and Abraham, S., Monitoring of the Launched Girder Bridge over the Iowa River on US 20, Iowa State University, Ames, IA, pp 100, (2004)

Wisconsin DOT (2013). WisDOT Bridge Manual, CH 7-Accelerated Bridge Construction, Wisconsin Department of Transportation, Madison, Wisconsin, pp 1-35.

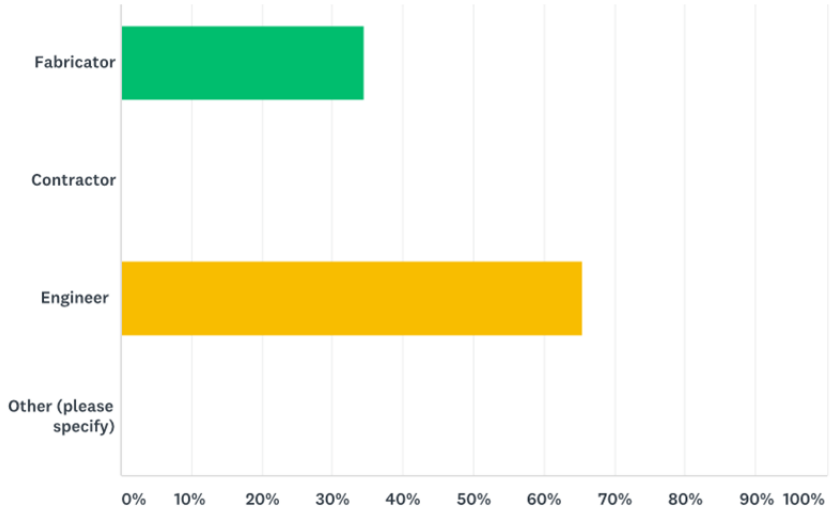
APPENDIX A

Results of Industry Questionnaire Regarding Proposed Tolerance Guidelines for Common Elements

A questionnaire was developed and was sent to each State Transportation agency, PCI Bridge producers, and industry experts. The goal of the questionnaire was to show the recommended tolerances for common bridge elements and ask for concurrence or recommended changes.

Q1 What is your occupation?

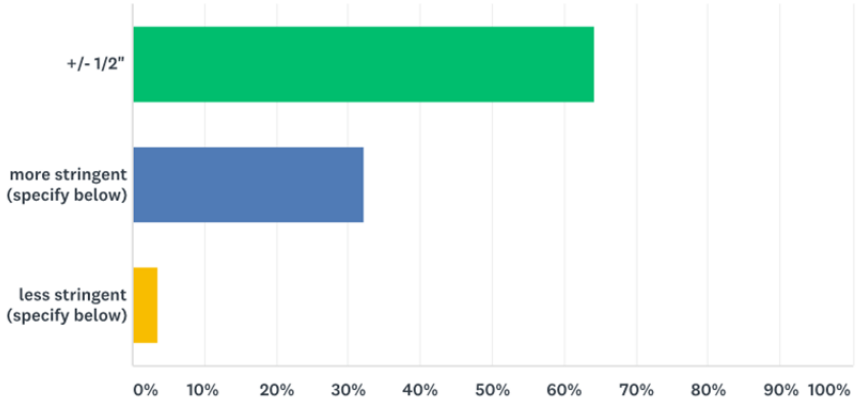
Answered: 29 Skipped: 0



ANSWER CHOICES	RESPONSES	
Fabricator	34.48%	10
Contractor	0.00%	0
Engineer	65.52%	19
Other (please specify)	0.00%	0
TOTAL		29

Q2 Column - Length (A):

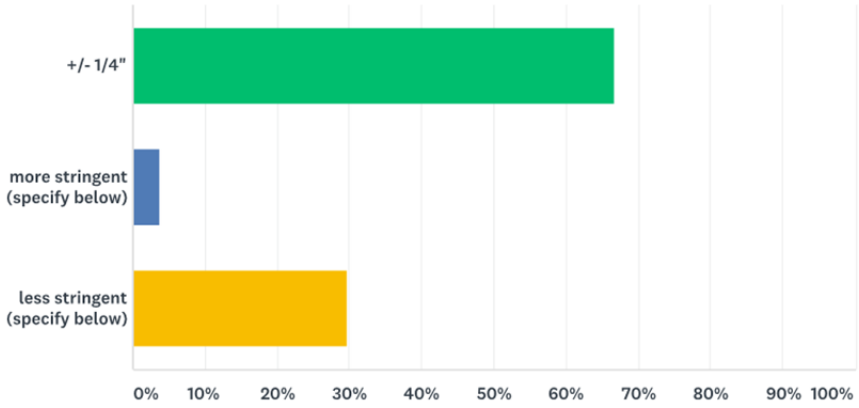
Answered: 28 Skipped: 1



ANSWER CHOICES	RESPONSES	
+/- 1/2"	64.29%	18
more stringent (specify below)	32.14%	9
less stringent (specify below)	3.57%	1
TOTAL		28

Q3 Footing - Length (A):

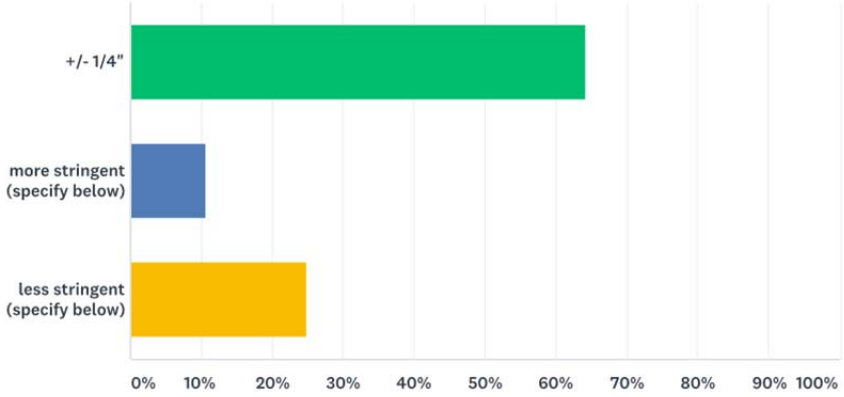
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4"	66.67%	18
more stringent (specify below)	3.70%	1
less stringent (specify below)	29.63%	8
TOTAL		27

Q4 Width (B):

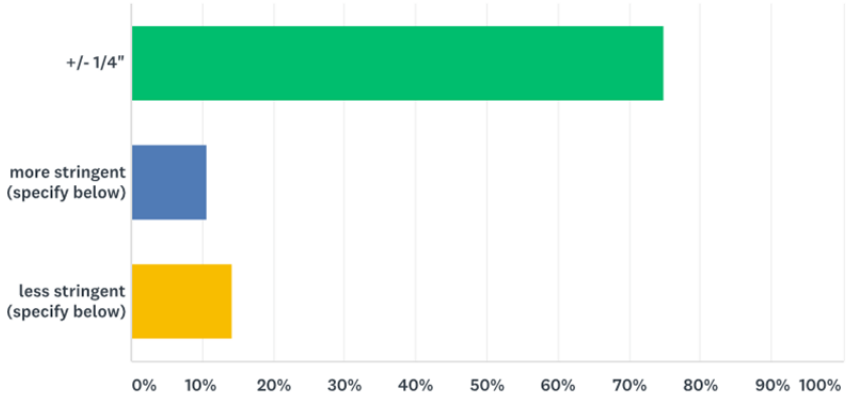
Answered: 28 Skipped: 1



ANSWER CHOICES	RESPONSES	
+/- 1/4"	64.29%	18
more stringent (specify below)	10.71%	3
less stringent (specify below)	25.00%	7
TOTAL		28

Q5 Depth (C):

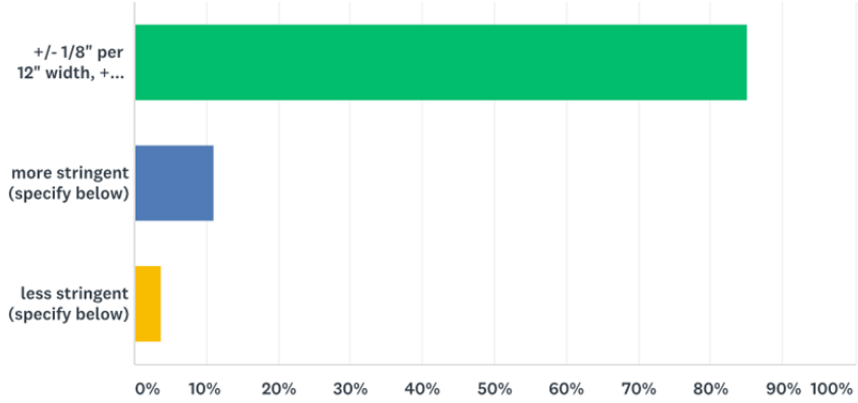
Answered: 28 Skipped: 1



ANSWER CHOICES	RESPONSES	
+/- 1/4"	75.00%	21
more stringent (specify below)	10.71%	3
less stringent (specify below)	14.29%	4
TOTAL		28

Q6 Column - Variation from specified squareness or skew (D):

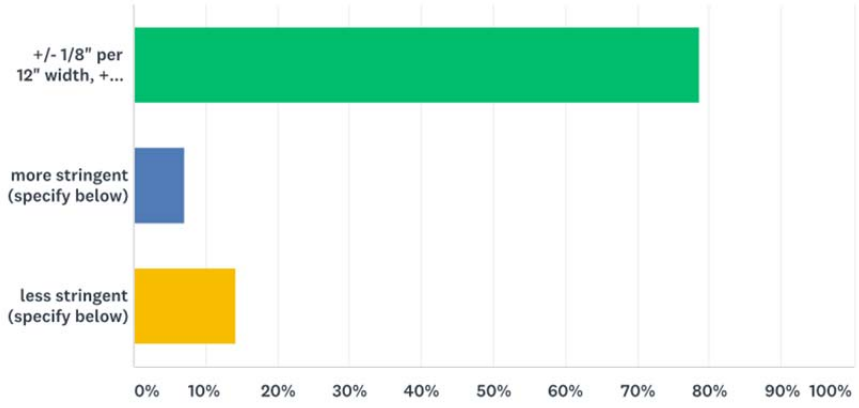
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12" width, +/- 3/8" maximum	85.19%	23
more stringent (specify below)	11.11%	3
less stringent (specify below)	3.70%	1
TOTAL		27

Q7 Footing - Variation from specified plan end squareness or skew (D):

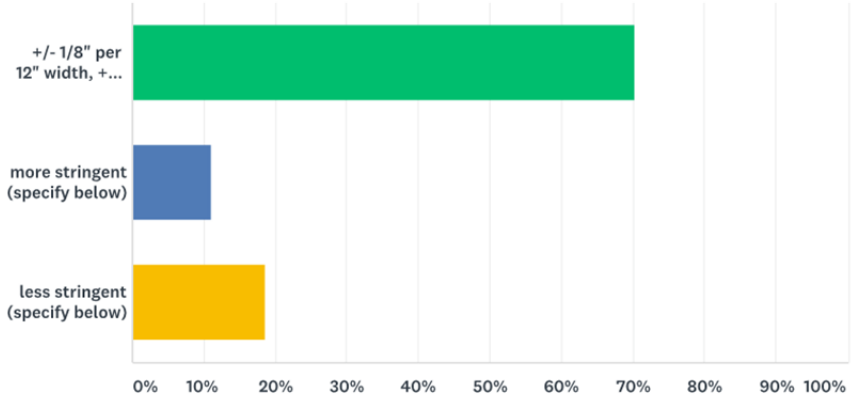
Answered: 28 Skipped: 1



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12" width, +/- 1/2" maximum	78.57%	22
more stringent (specify below)	7.14%	2
less stringent (specify below)	14.29%	4
TOTAL		28

Q8 Footing - Variation from specified plan end squareness or skew (E):

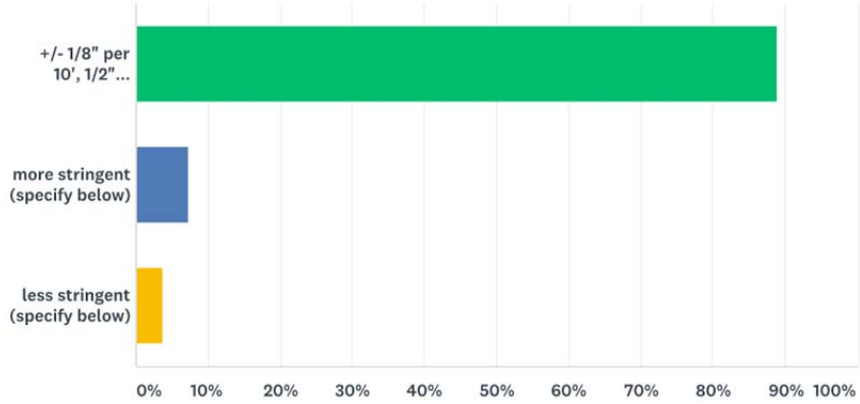
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12" width, +/- 1/2" maximum	70.37%	19
more stringent (specify below)	11.11%	3
less stringent (specify below)	18.52%	5
TOTAL		27

Q9 Column - Sweep (F):

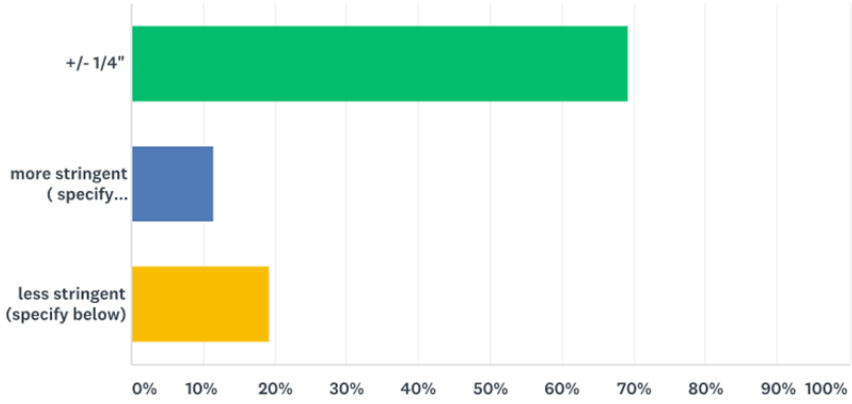
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 10', 1/2" maximum	88.89%	24
more stringent (specify below)	7.41%	2
less stringent (specify below)	3.70%	1
TOTAL		27

Q10 Location of grouted splice coupler or dowels (G):

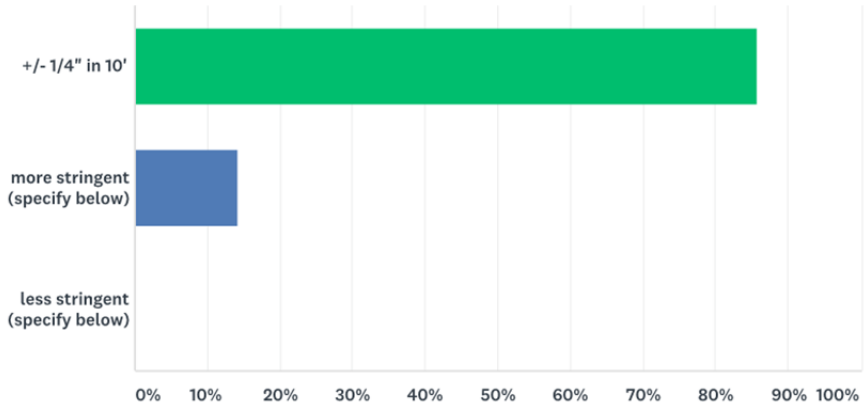
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	69.23%	18
more stringent (specify below)	11.54%	3
less stringent (specify below)	19.23%	5
TOTAL		26

Q11 Local smoothness of any surface (H):

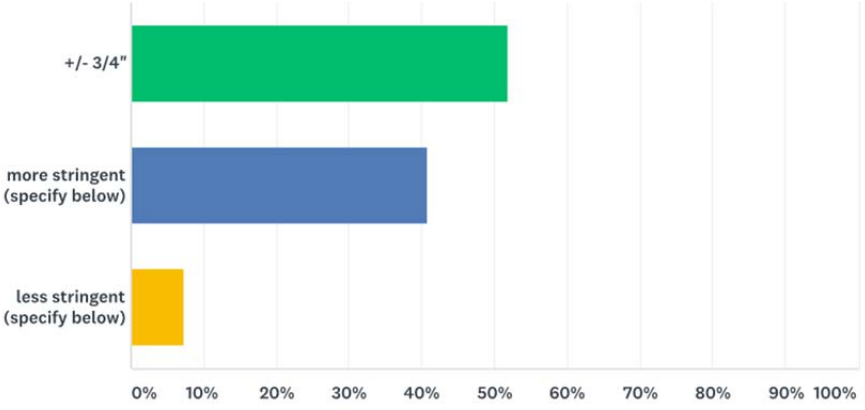
Answered: 28 Skipped: 1



ANSWER CHOICES	RESPONSES	
+/- 1/4" in 10'	85.71%	24
more stringent (specify below)	14.29%	4
less stringent (specify below)	0.00%	0
TOTAL		28

Q12 Pier or Wall Cap - Length (A):

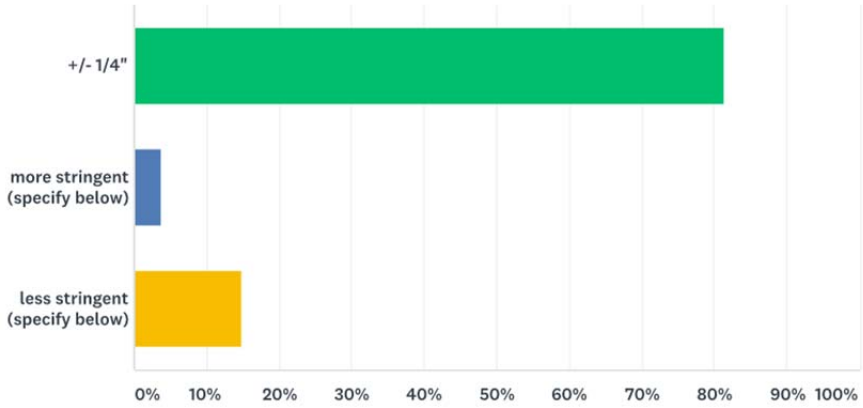
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 3/4"	51.85%	14
more stringent (specify below)	40.74%	11
less stringent (specify below)	7.41%	2
TOTAL		27

Q13 Wall Panel - Length (A):

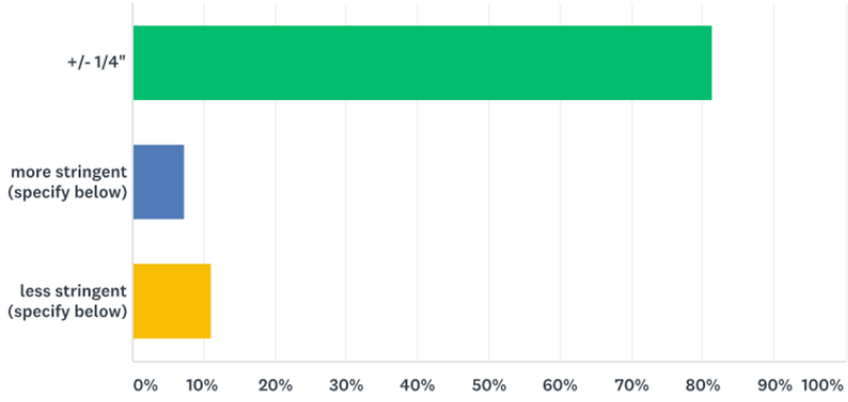
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4"	81.48%	22
more stringent (specify below)	3.70%	1
less stringent (specify below)	14.81%	4
TOTAL		27

Q14 Width (B):

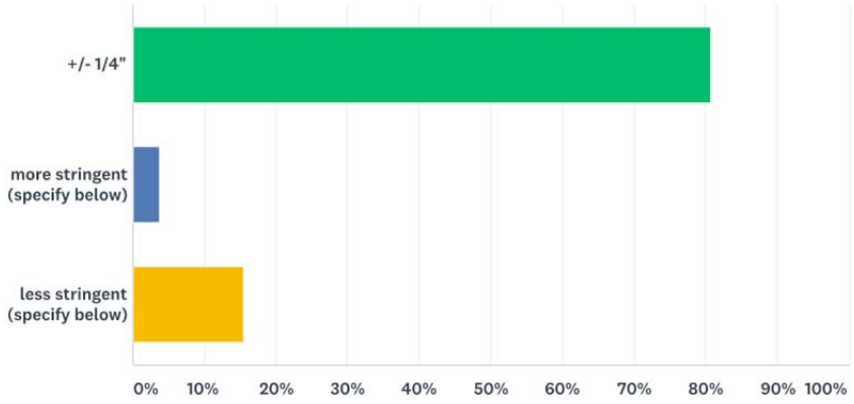
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4"	81.48%	22
more stringent (specify below)	7.41%	2
less stringent (specify below)	11.11%	3
TOTAL		27

Q15 Depth (C):

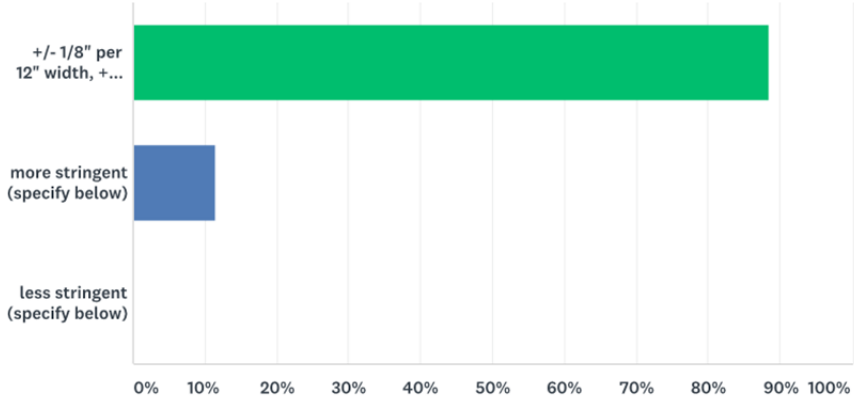
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	80.77%	21
more stringent (specify below)	3.85%	1
less stringent (specify below)	15.38%	4
TOTAL		26

Q16 Variation from specified plan end squareness or skew (D):

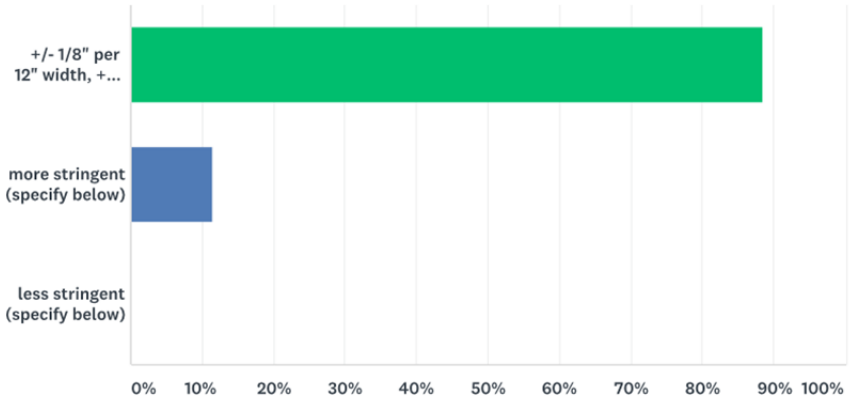
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12" width, +/- 1/2" maximum	88.46%	23
more stringent (specify below)	11.54%	3
less stringent (specify below)	0.00%	0
TOTAL		26

Q17 Variation from specified elevation end squareness or skew (E):

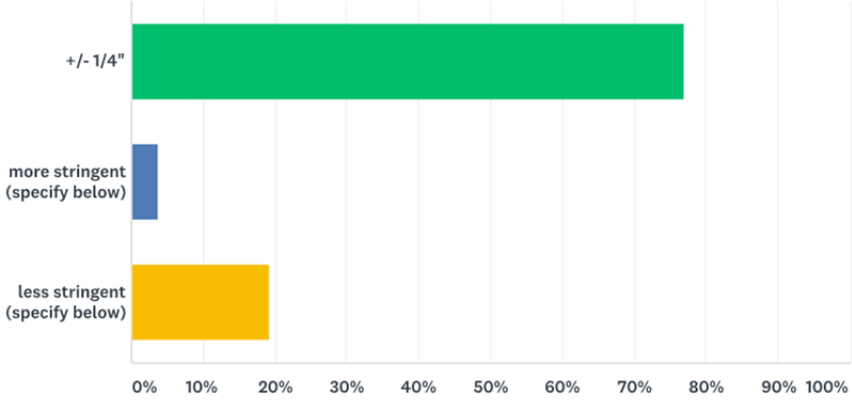
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12" width, +/- 1/2" maximum	88.46%	23
more stringent (specify below)	11.54%	3
less stringent (specify below)	0.00%	0
TOTAL		26

Q18 Pier or Wall Cap - Sweep if prestressed (F) for member length up to 40 feet:

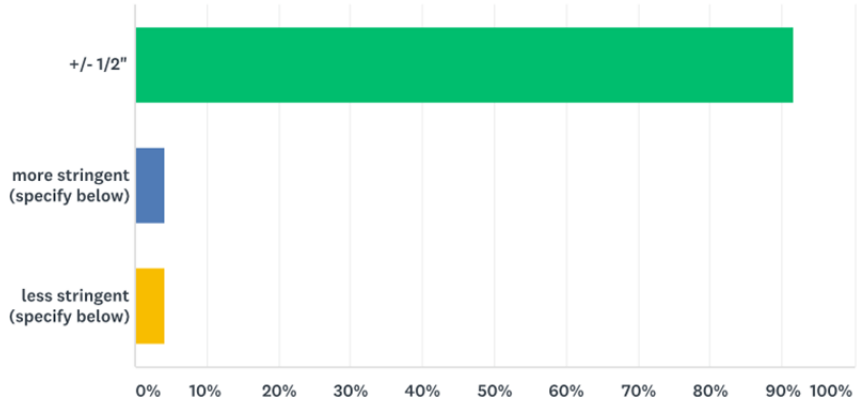
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	76.92%	20
more stringent (specify below)	3.85%	1
less stringent (specify below)	19.23%	5
TOTAL		26

Q19 Pier or Wall Cap - Sweep if prestressed (F) for member length between 40 feet and 60 feet:

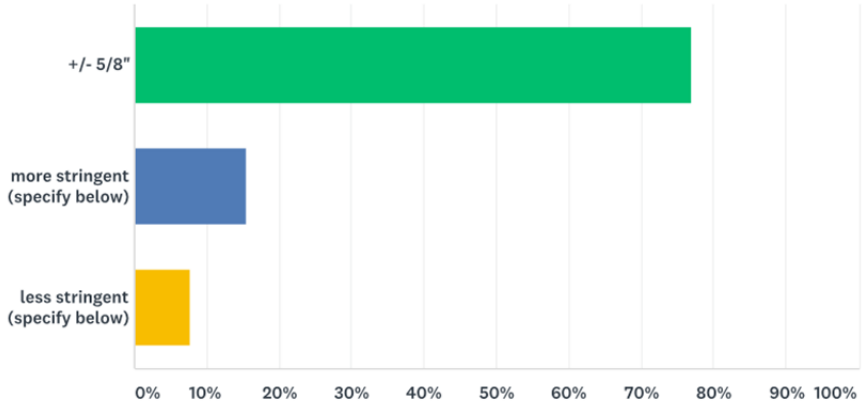
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1/2"	91.67%	22
more stringent (specify below)	4.17%	1
less stringent (specify below)	4.17%	1
TOTAL		24

Q20 Pier or Wall Cap - Sweep if prestressed (F) for member length over 60 feet:

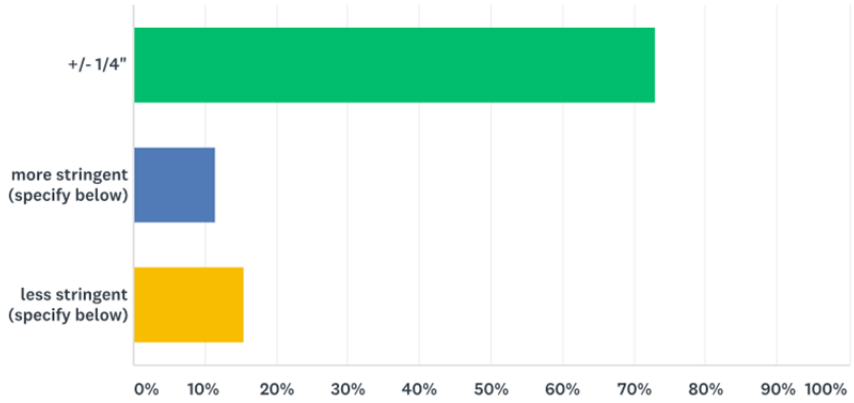
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 5/8"	76.92%	20
more stringent (specify below)	15.38%	4
less stringent (specify below)	7.69%	2
TOTAL		26

Q21 Location of grouted splice coupler (G):

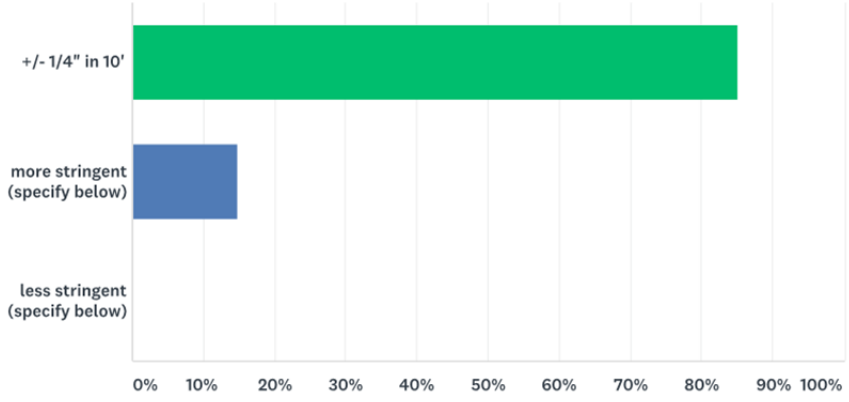
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	73.08%	19
more stringent (specify below)	11.54%	3
less stringent (specify below)	15.38%	4
TOTAL		26

Q22 Local smoothness (H):

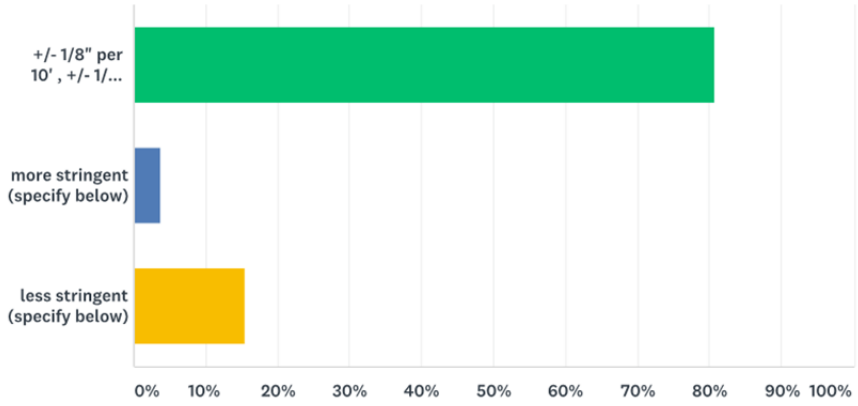
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4" in 10'	85.19%	23
more stringent (specify below)	14.81%	4
less stringent (specify below)	0.00%	0
TOTAL		27

Q23 Pier or Wall Cap - Variation from specified camber if prestressed (J):

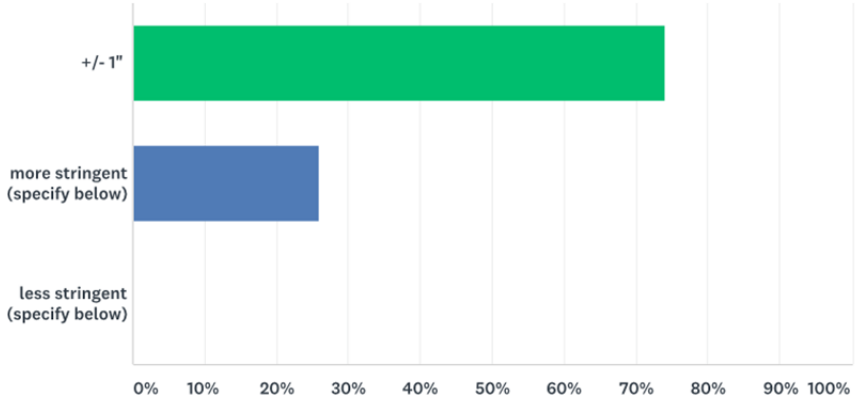
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 10' , +/- 1/2" maximum	80.77%	21
more stringent (specify below)	3.85%	1
less stringent (specify below)	15.38%	4
TOTAL		26

Q24 Location of pocket, blockout, or void (P):

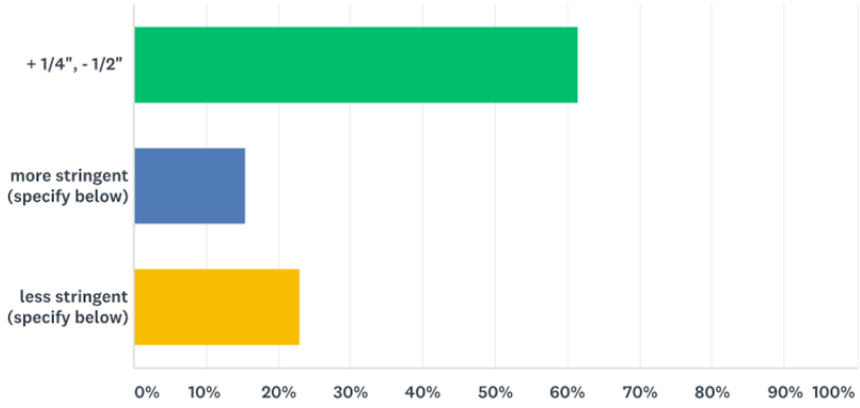
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1"	74.07%	20
more stringent (specify below)	25.93%	7
less stringent (specify below)	0.00%	0
TOTAL		27

Q25 Stirrup projection from surface (S3):

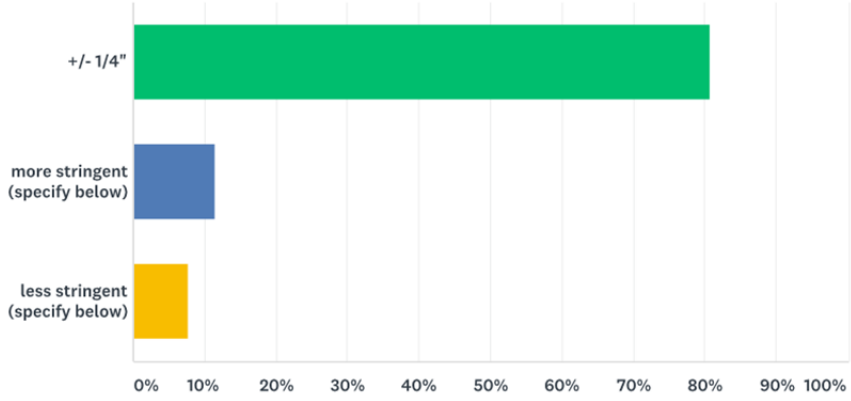
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+ 1/4", - 1/2"	61.54%	16
more stringent (specify below)	15.38%	4
less stringent (specify below)	23.08%	6
TOTAL		26

Q26 Full Depth Deck Panel & Approach Slab - Length (A):

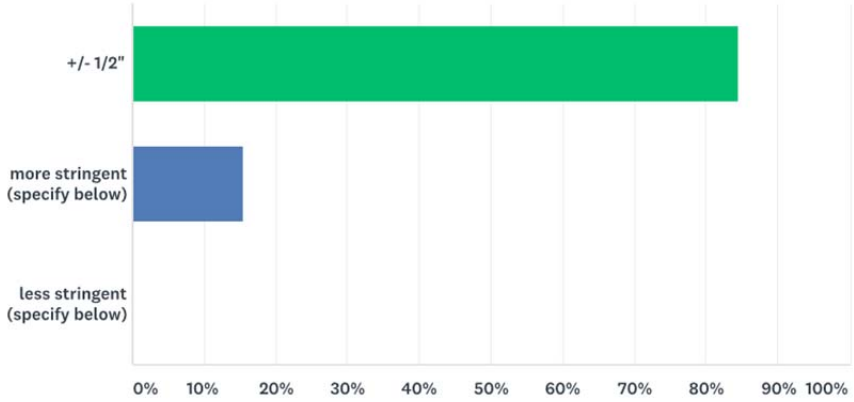
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	80.77%	21
more stringent (specify below)	11.54%	3
less stringent (specify below)	7.69%	2
TOTAL		26

Q27 Sleeper Slab - Length (A):

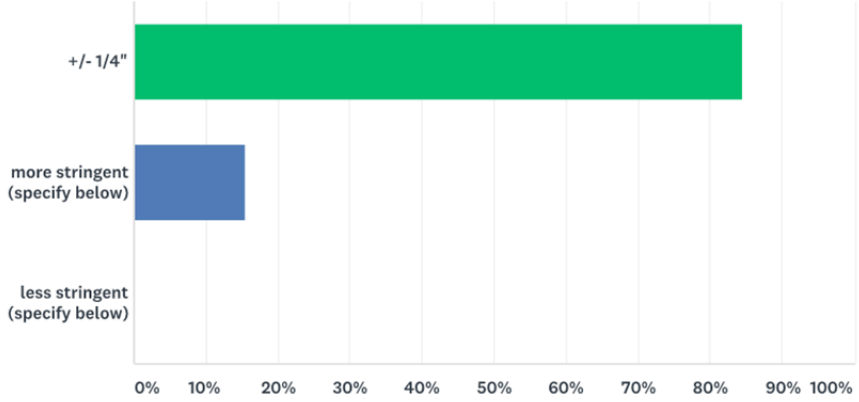
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/2"	84.62%	22
more stringent (specify below)	15.38%	4
less stringent (specify below)	0.00%	0
TOTAL		26

Q28 Width (B):

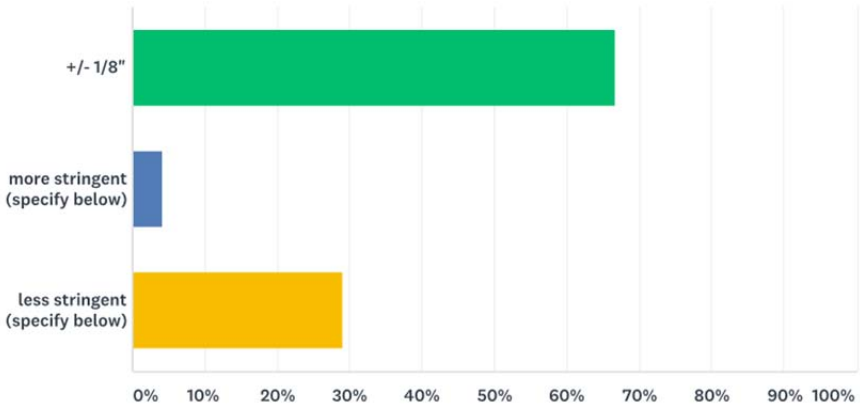
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	84.62%	22
more stringent (specify below)	15.38%	4
less stringent (specify below)	0.00%	0
TOTAL		26

Q29 Full Depth Deck Panel - Depth (C):

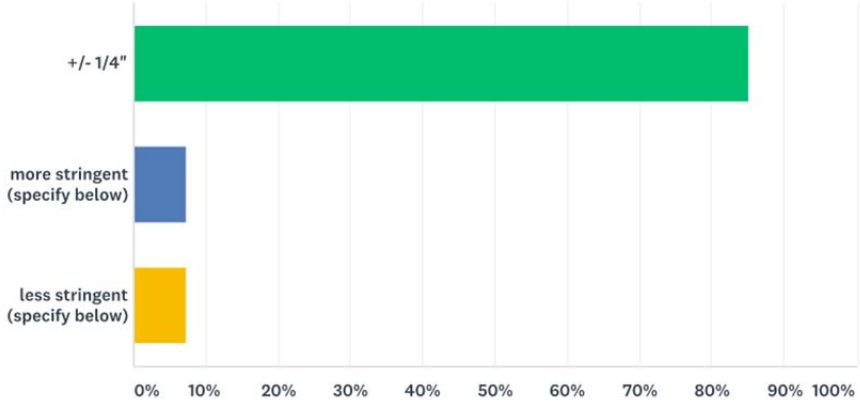
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1/8"	66.67%	16
more stringent (specify below)	4.17%	1
less stringent (specify below)	29.17%	7
TOTAL		24

Q30 Approach & Sleeper Slab - Depth (C):

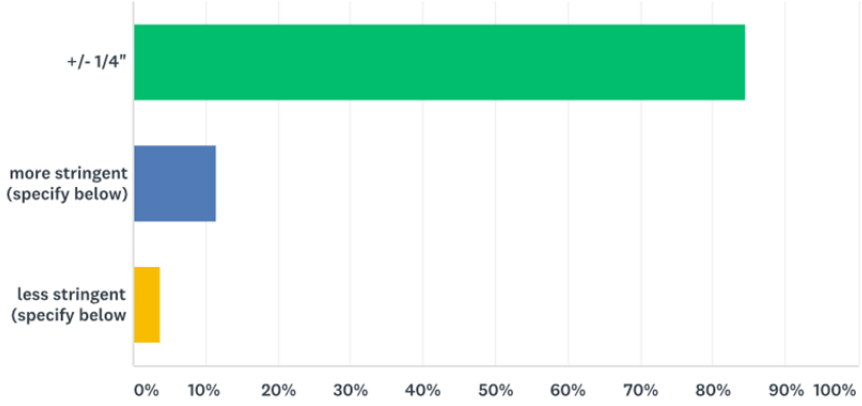
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4"	85.19%	23
more stringent (specify below)	7.41%	2
less stringent (specify below)	7.41%	2
TOTAL		27

Q31 Full Depth Deck Panel - Variation from specified span end squareness or skew (D):

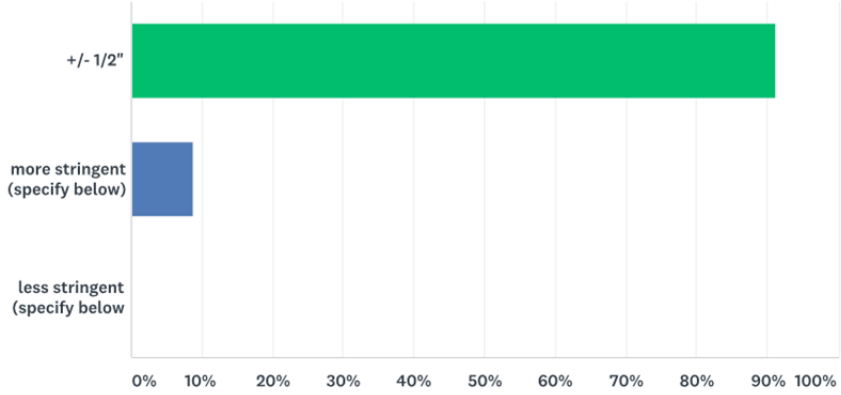
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	84.62%	22
more stringent (specify below)	11.54%	3
less stringent (specify below	3.85%	1
TOTAL		26

Q32 Approach Slab - Variation from specified span end squareness or skew (D):

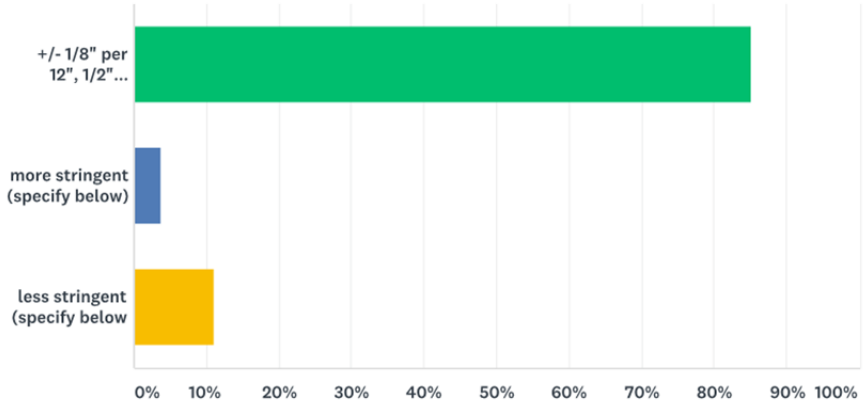
Answered: 23 Skipped: 6



ANSWER CHOICES	RESPONSES	
+/- 1/2"	91.30%	21
more stringent (specify below)	8.70%	2
less stringent (specify below	0.00%	0
TOTAL		23

Q33 Sleeper Slab - Variation from specified span end squareness or skew (D):

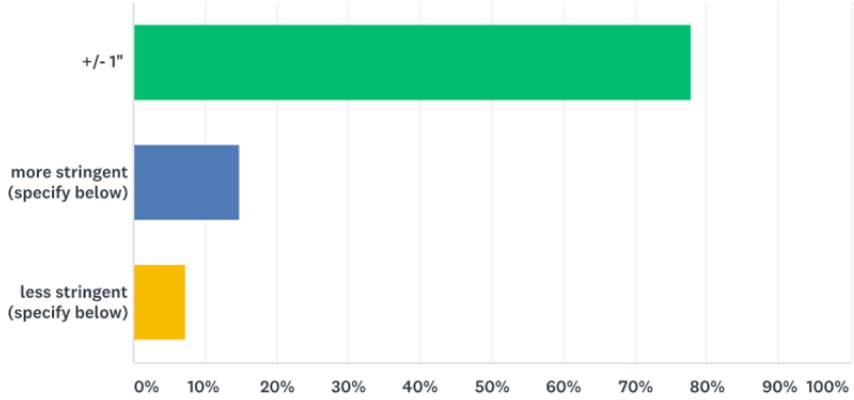
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/8" per 12", 1/2" maximum	85.19%	23
more stringent (specify below)	3.70%	1
less stringent (specify below	11.11%	3
TOTAL		27

Q34 Location of leveling device (E):

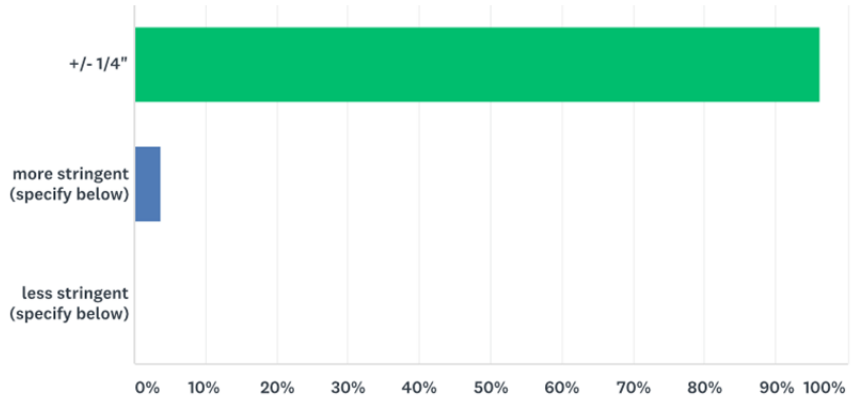
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1"	77.78%	21
more stringent (specify below)	14.81%	4
less stringent (specify below)	7.41%	2
TOTAL		27

Q35 Full Depth Deck Panel - Sweep over member length (F):

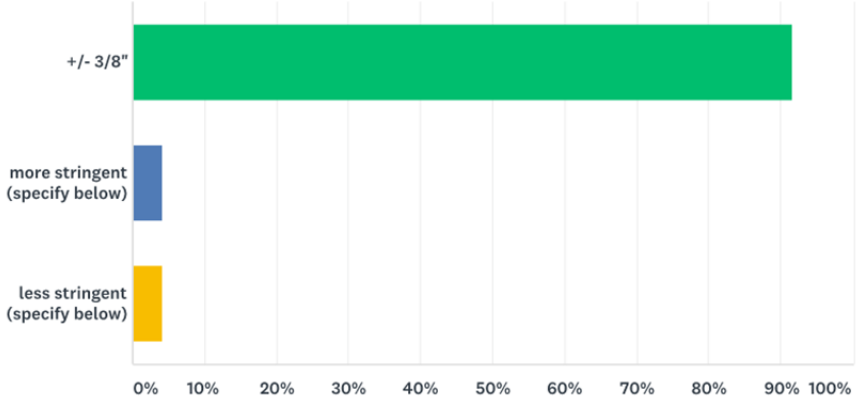
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	96.15%	25
more stringent (specify below)	3.85%	1
less stringent (specify below)	0.00%	0
TOTAL		26

Q36 Approach Slab - Sweep over member length (F):

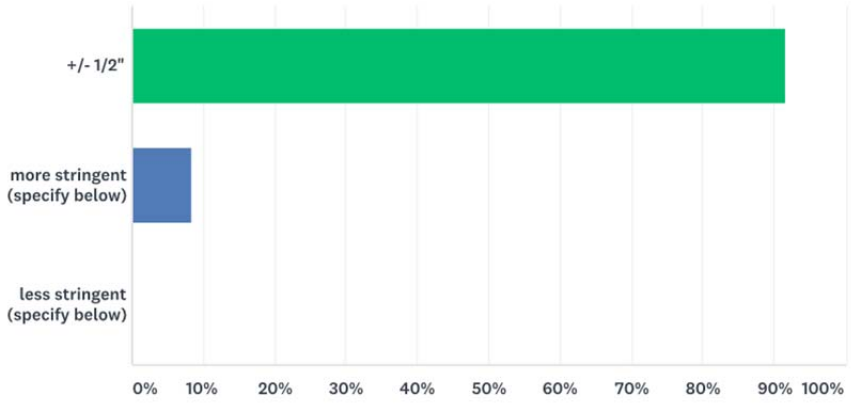
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 3/8"	91.67%	22
more stringent (specify below)	4.17%	1
less stringent (specify below)	4.17%	1
TOTAL		24

Q37 Sleeper Slab - Sweep over member length (F):

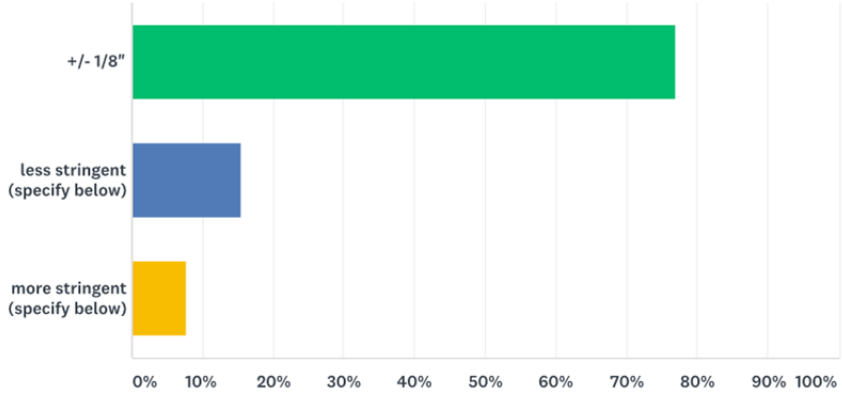
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1/2"	91.67%	22
more stringent (specify below)	8.33%	2
less stringent (specify below)	0.00%	0
TOTAL		24

Q38 Full Depth Deck Panel - Distance from common working point to center of any PT duct (G):

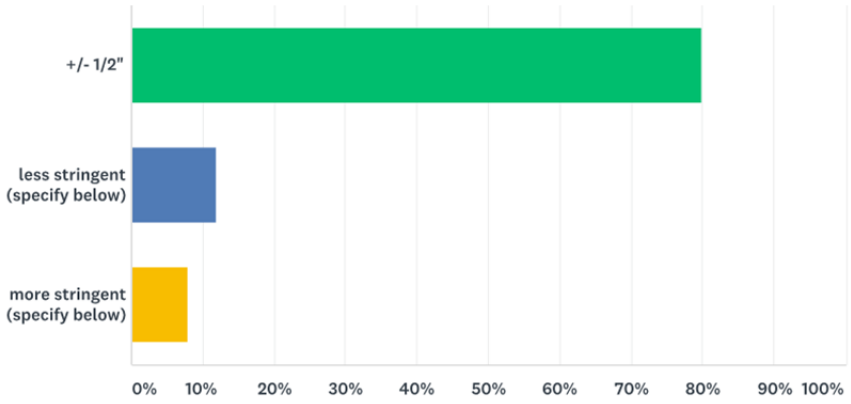
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/8"	76.92%	20
less stringent (specify below)	15.38%	4
more stringent (specify below)	7.69%	2
TOTAL		26

Q39 Approach Slab - Location of projecting reinforcement (G):

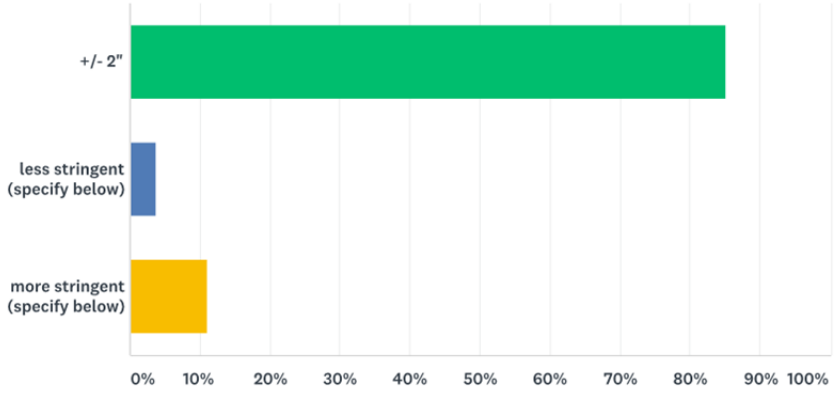
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/2"	80.00%	20
less stringent (specify below)	12.00%	3
more stringent (specify below)	8.00%	2
TOTAL		25

Q40 Sleeper Slab - Location of leveling device or grout port (G):

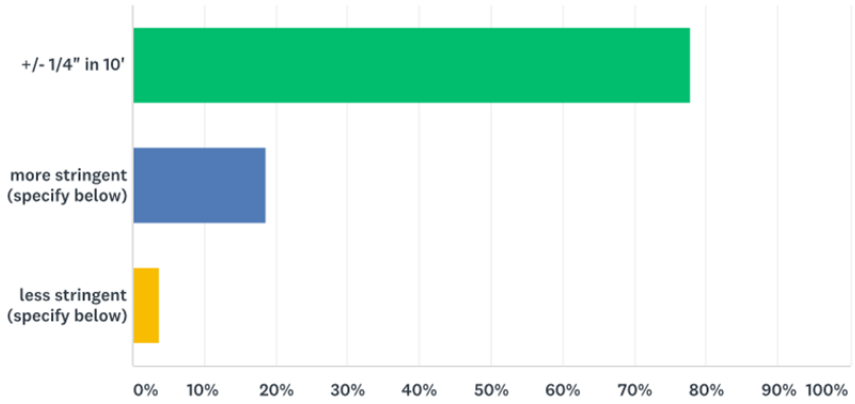
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 2"	85.19%	23
less stringent (specify below)	3.70%	1
more stringent (specify below)	11.11%	3
TOTAL		27

Q41 Full Depth Deck Panel - Local smoothness of any surface (H):

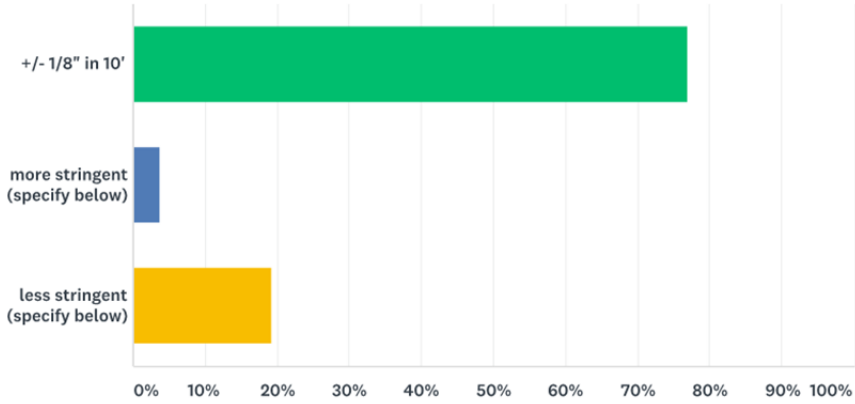
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/4" in 10'	77.78%	21
more stringent (specify below)	18.52%	5
less stringent (specify below)	3.70%	1
TOTAL		27

Q42 Approach & Sleeper Slab - Local smoothness of any surface (H):

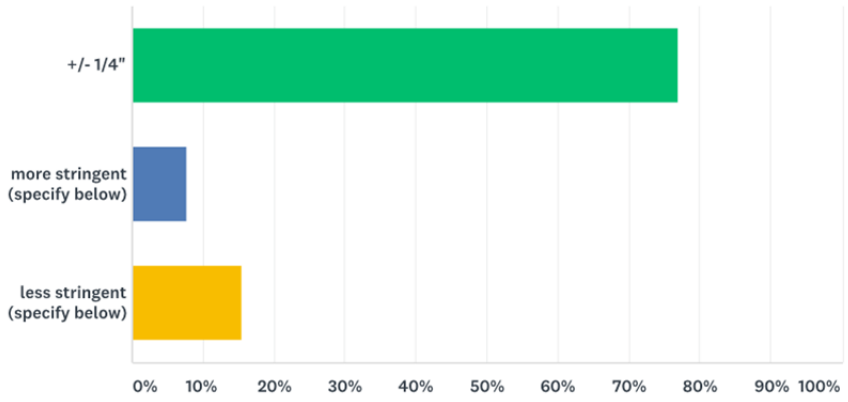
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/8" in 10'	76.92%	20
more stringent (specify below)	3.85%	1
less stringent (specify below)	19.23%	5
TOTAL		26

Q43 Full Depth Deck Panel - Camber variation from design camber (J):

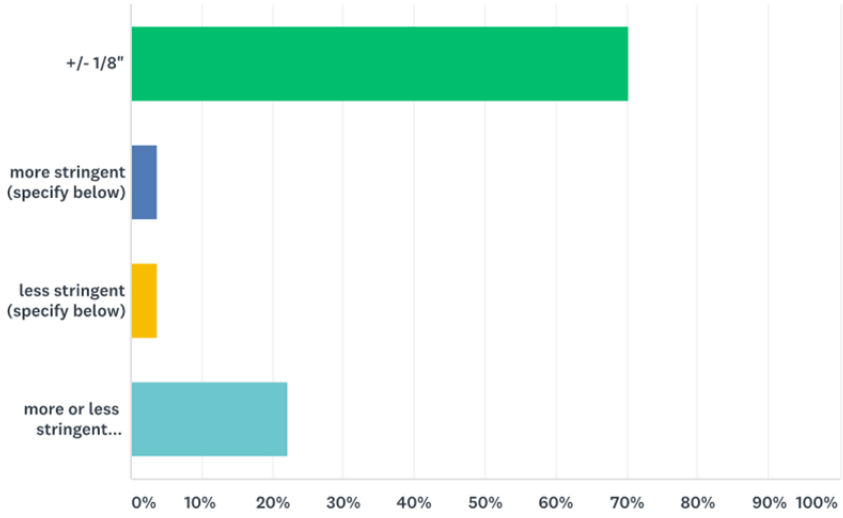
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	76.92%	20
more stringent (specify below)	7.69%	2
less stringent (specify below)	15.38%	4
TOTAL		26

Q44 Full Depth Deck Panel - Center of PT duct at edge of slab (K):

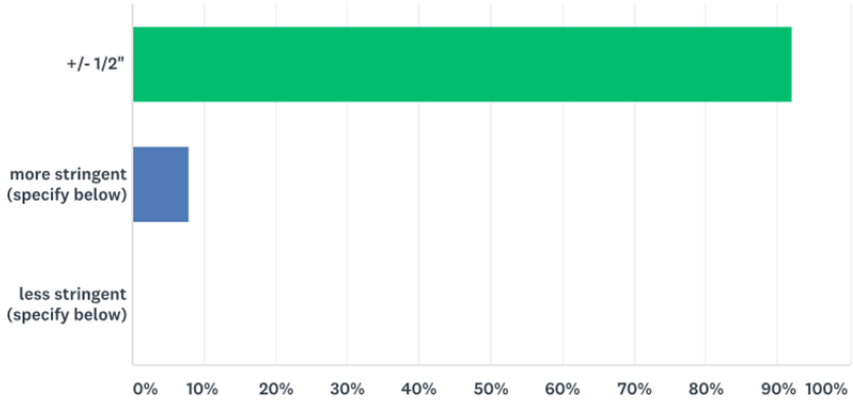
Answered: 27 Skipped: 2



ANSWER CHOICES	RESPONSES	
+/- 1/8"	70.37%	19
more stringent (specify below)	3.70%	1
less stringent (specify below)	3.70%	1
more or less stringent (please specify)	22.22%	6
TOTAL		27

Q45 Location of shear connector pocket or blockout (P):

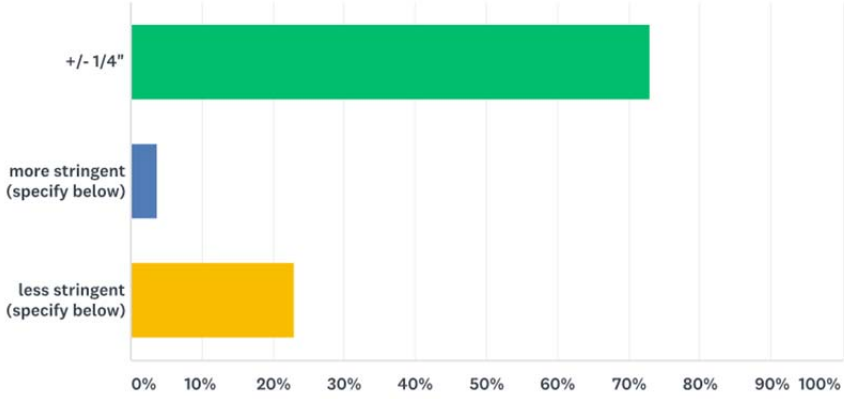
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/2"	92.00%	23
more stringent (specify below)	8.00%	2
less stringent (specify below)	0.00%	0
TOTAL		25

Q46 Approach Slab - Projection length of reinforcement (Q):

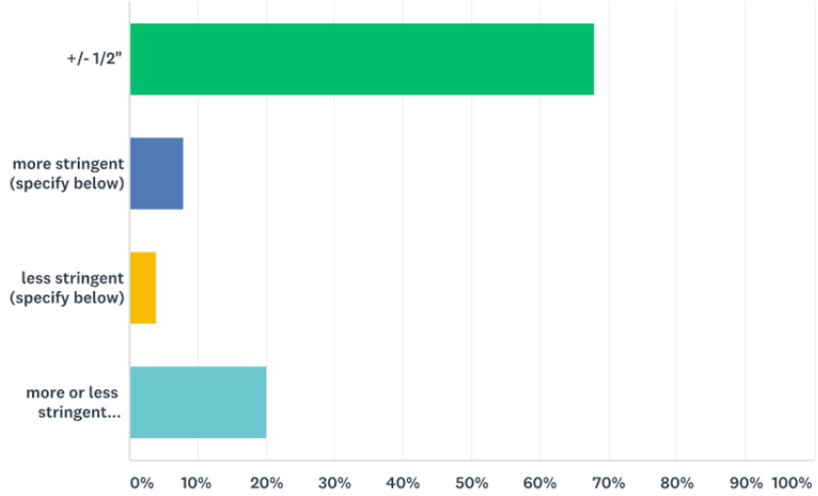
Answered: 26 Skipped: 3



ANSWER CHOICES	RESPONSES	
+/- 1/4"	73.08%	19
more stringent (specify below)	3.85%	1
less stringent (specify below)	23.08%	6
TOTAL		26

Q47 Plan location from working line (P):

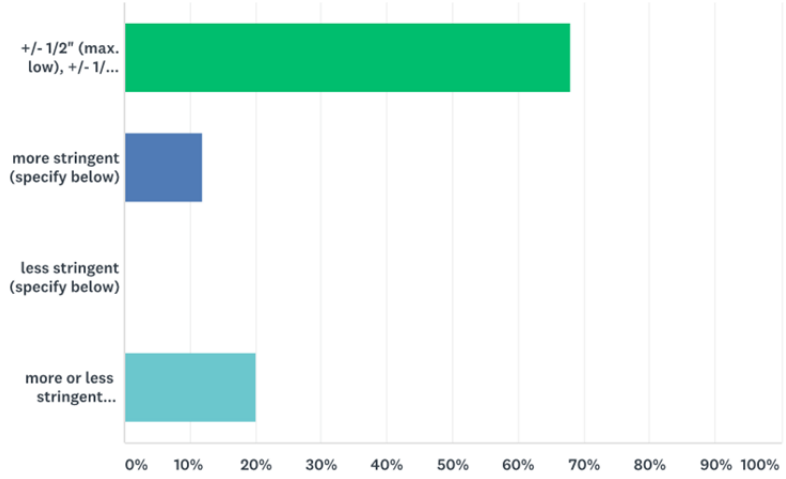
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/2"	68.00%	17
more stringent (specify below)	8.00%	2
less stringent (specify below)	4.00%	1
more or less stringent (please specify)	20.00%	5
TOTAL		25

Q48 Top of elevation from nominal top elevation (L):

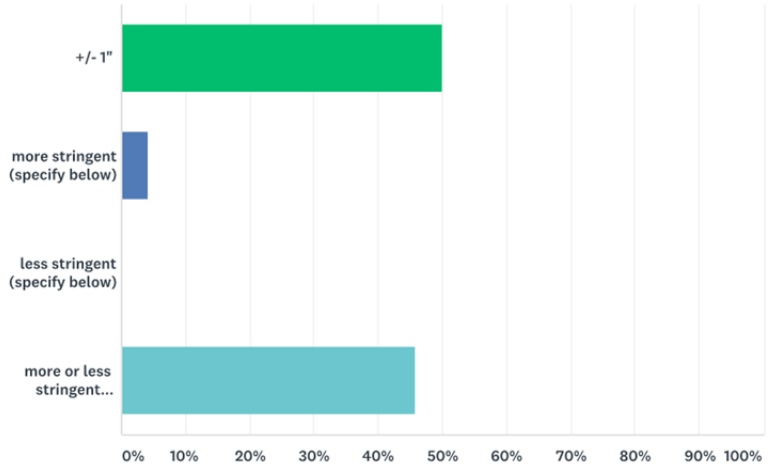
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/2" (max. low), +/- 1/4" (max. high)	68.00%	17
more stringent (specify below)	12.00%	3
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	20.00%	5
TOTAL		25

Q49 Column - Maximum plumb variation over height of column (R):

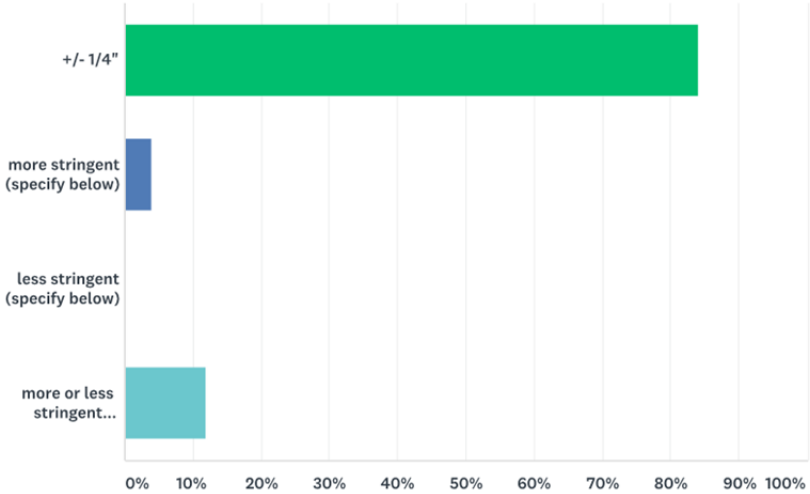
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1"	50.00%	12
more stringent (specify below)	4.17%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	45.83%	11
TOTAL		24

Q50 Column - Maximum plumb variation in any 10 feet (U):

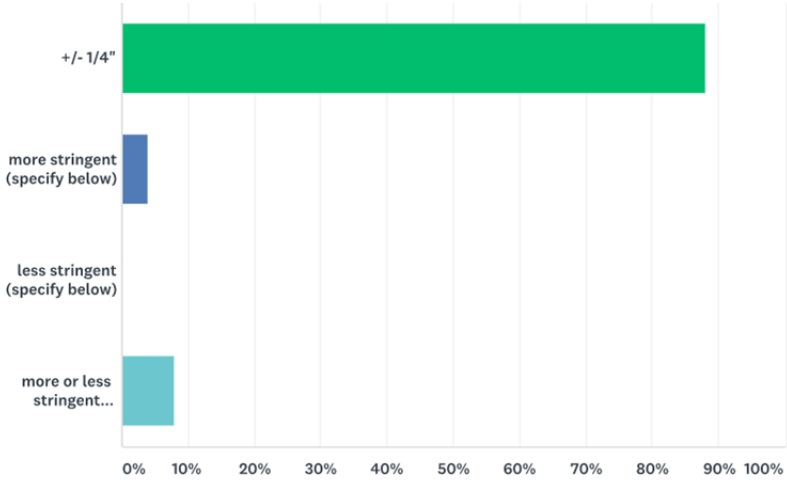
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/4"	84.00%	21
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	12.00%	3
TOTAL		25

Q51 Column - Jog in alignment of matching edges (V):

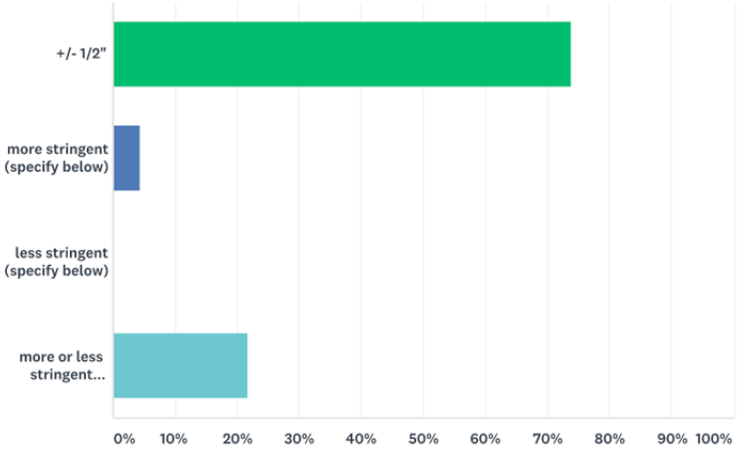
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/4"	88.00%	22
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	8.00%	2
TOTAL		25

Q52 Plan location from working line (P):

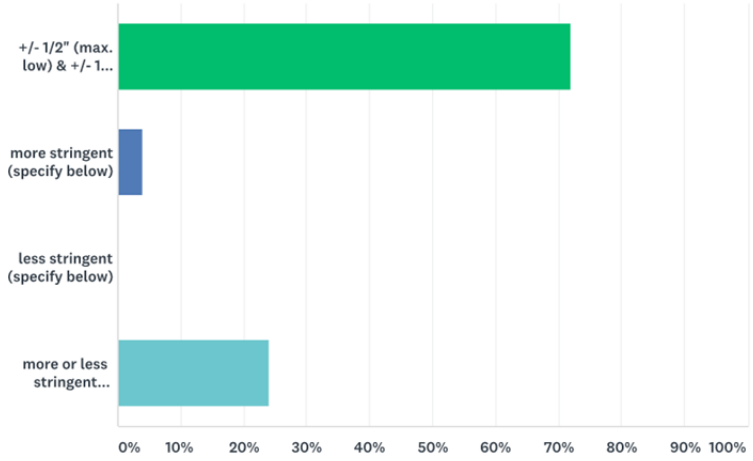
Answered: 23 Skipped: 6



ANSWER CHOICES	RESPONSES	
+/- 1/2"	73.91%	17
more stringent (specify below)	4.35%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	21.74%	5
TOTAL		23

Q53 Top elevation from nominal top elevation (L):

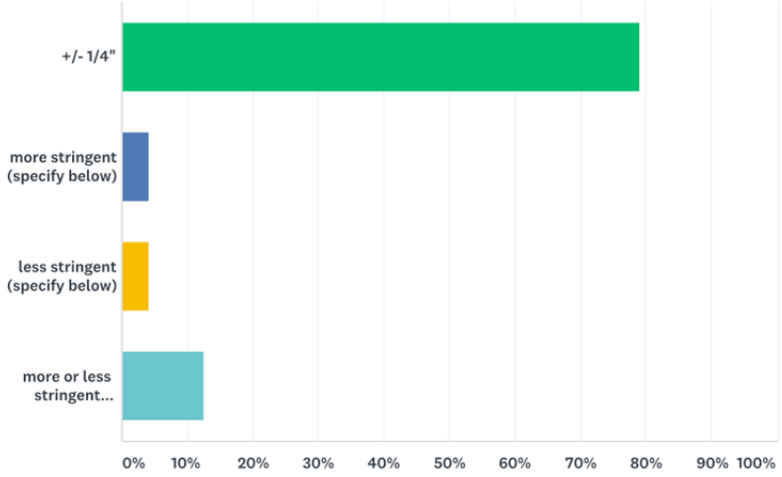
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/2" (max. low) & +/- 1/4" (max. high)	72.00%	18
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	24.00%	6
TOTAL		25

Q54 Maximum plumb variation in any 10 feet (U):

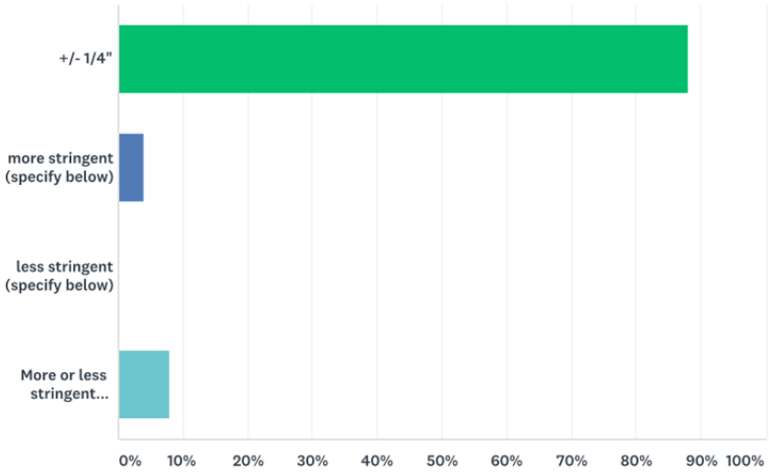
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1/4"	79.17%	19
more stringent (specify below)	4.17%	1
less stringent (specify below)	4.17%	1
more or less stringent (please specify)	12.50%	3
TOTAL		24

Q55 Jog in alignment of matching edges (V):

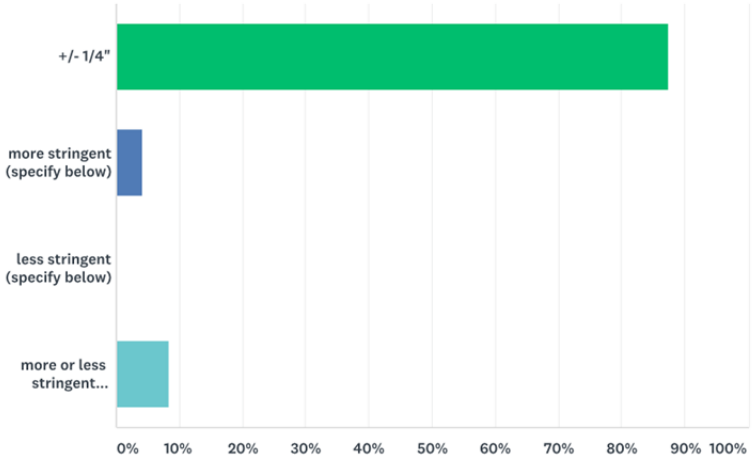
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/4"	88.00%	22
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
More or less stringent (please specify)	8.00%	2
TOTAL		25

Q56 Plan location from working line (P):

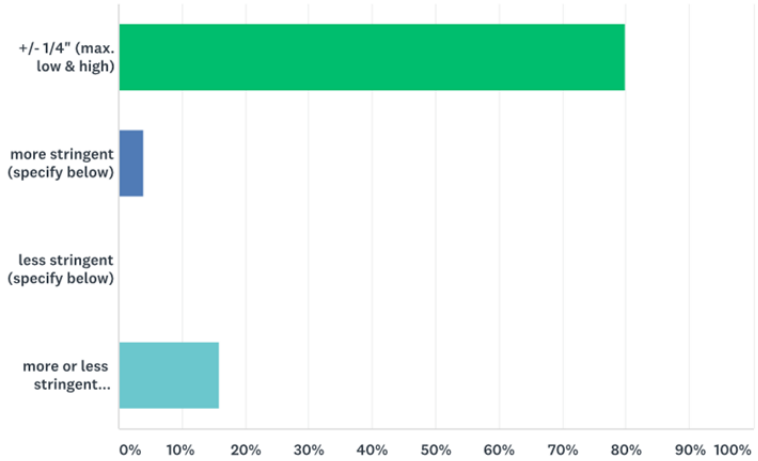
Answered: 24 Skipped: 5



ANSWER CHOICES	RESPONSES	
+/- 1/4"	87.50%	21
more stringent (specify below)	4.17%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	8.33%	2
TOTAL		24

Q57 Top elevation from nominal top elevation (L) with thick overlay:

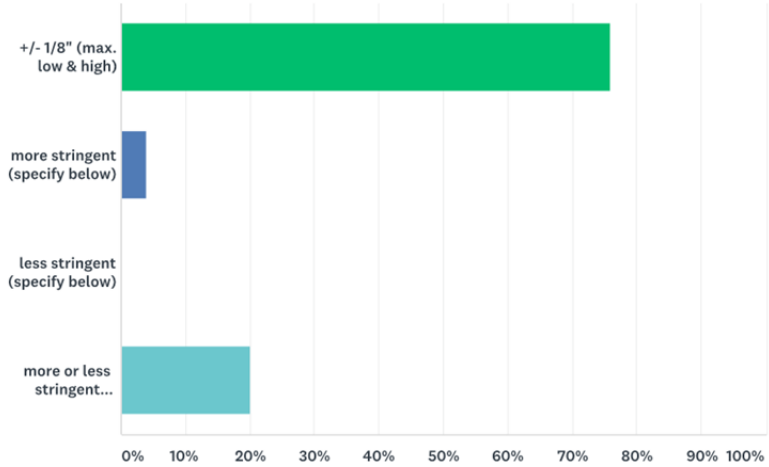
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/4" (max. low & high)	80.00%	20
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	16.00%	4
TOTAL		25

Q58 Top elevation from nominal top elevation (L) with thin overlay or bare deck combined with grinding:

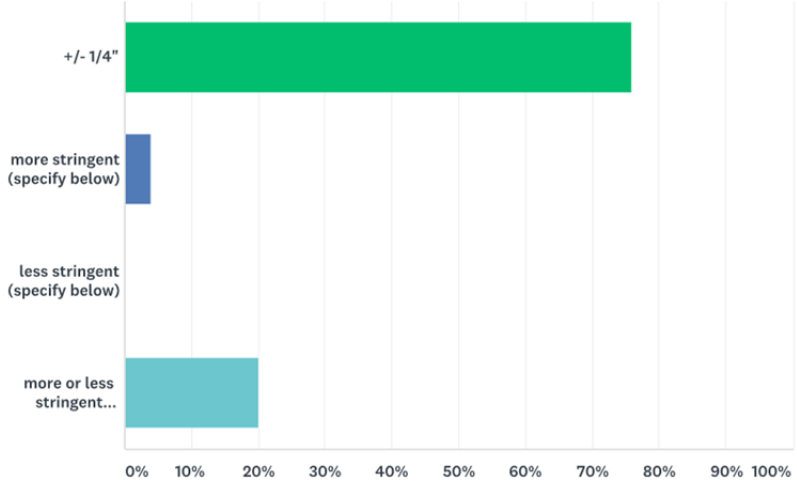
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/8" (max. low & high)	76.00%	19
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	20.00%	5
TOTAL		25

Q59 Differential top elevation (R) with thick overlay:

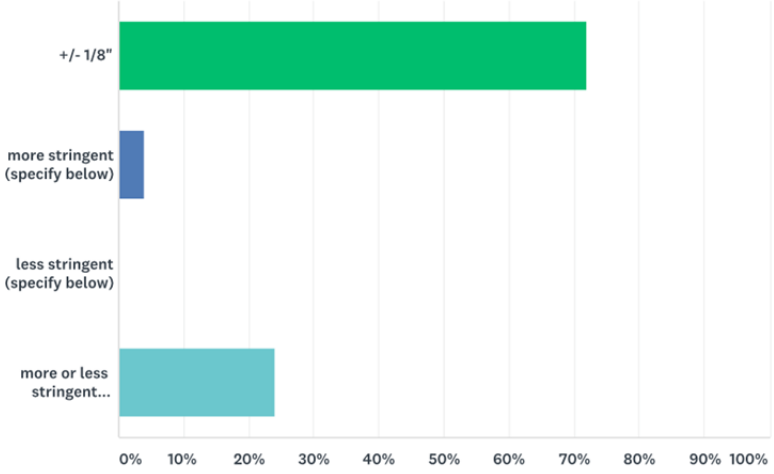
Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/4"	76.00%	19
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	20.00%	5
TOTAL		25

Q60 Differential top elevation (R) with thin overlay or bare deck combined with grinding:

Answered: 25 Skipped: 4



ANSWER CHOICES	RESPONSES	
+/- 1/8"	72.00%	18
more stringent (specify below)	4.00%	1
less stringent (specify below)	0.00%	0
more or less stringent (please specify)	24.00%	6
TOTAL		25

APPENDIX B

Statistical Analysis of Multi-Variable Equations

The following notes contain the step-by-step process that was used for the statistical analysis of the joint width equations contained in the proposed tolerance guideline.

Step 1: Determine Which Tolerances Will Affect Joint Width

1. Vertical Joints (side-by-side elements)
    a. Fabrication Tolerances – Length (A), local smoothness (H), variation from specified end squareness or skew (D)
    b. Horizontal Erection Tolerances (L)
    c. Assumptions
        i. Half of the length tolerances of the two adjacent elements are used based on the assumption that half of the additional length or width is applied to each end of the element (half to the right joint and half to the left joint).
        ii. The fabrication tolerances, variation from specified end squareness/skew and local smoothness of any surface, occur at different locations along the joint; therefore they are not additive. In order to be conservative the maximum of the two will be chosen. MAX function was used in the Risk Analyzer input file. This resulted in a different output then when user chose maximum for each situation. The MAX function allows the program to consider the possibility that one tolerance may sometimes be greater than the other even if the maximum tolerance is less.
        iii. Half of the erection tolerance is used. This assumes one of the two side-by-side elements are reset and the other is erected within tolerance; therefore erection tolerance to be reduced by two.
            1. Instead of setting one element tolerance to zero and including the maximum possible tolerance of the other element, half of both maximum tolerances were included in the Risk Analyzer input. Result of setting one side to zero is a larger tolerance factor therefore a larger modified tolerance since maximum tolerance is the same. Chose half erection tolerance to limit size of joint.
            2. Researched output for assuming no elements are reset as well as all elements are reset but in order to narrow down risk analyzer runs moved forward with assuming one of the two elements are reset.
2. Horizontal Joints (stacked elements)
    a. Fabrication Tolerances – Height/Depth (A/C), local smoothness (H), variation from specified end squareness or skew (D), variance from specified camber-prestressed only (J)
        i. Sweep is not included in joint tolerances. Lateral camber or sweep is a variation in member horizontal alignment. This can be caused by horizontal eccentric prestress

in narrow members. Typically in long slender precast elements such as columns and caps. Ignored because columns and caps do not have joints along lateral edge where the tolerance would affect the joint width.

b. Vertical Elevation Tolerances (L)
c. Prestressed Only – variance from specified camber (J)
d. Assumptions
    i. Lower element is installed within tolerance and the entire upper element height or depth tolerance is used. Therefore the lower height or depth fabrication tolerance can be ignored.
    ii. Variation from specified end squareness/skew fabrication tolerance for wall panels are accounted for in the side-by-side element joint width tolerance and are ignored in stacked element joint thickness tolerance.
        1. For joints between panel & cap and panel & footing, skew applied to cap and footing but not at joints between column & cap and column & footing. This is because columns are typically narrow enough to not be affected by the horizontal member skew. When skew of cap and footing over the entire length is in consideration it needs to be accounted for.
    iii. The fabrication tolerances, variation from specified end squareness/skew and local smoothness of any surface, can occur at the same location along the joint; therefore they can be additive.
        1. Variation from specified end squareness/skew only occurs at columns
        2. Local smoothness is assumed to only occur at wide elements which includes Wall Panels, Caps, and Footings
        3. Skew and smoothness are additive for horizontal joints (column to horizontal element only) and not for vertical joints because smoothness variation on a skewed end will most likely not affect minimum joint opening.
    iv. Upper element elevation tolerances are assumed to be applied to top of element.
        1. If there is a joint above upper element – the upper joint will account for upper element elevation tolerance
        2. If there is no joint above upper element – the top elevation from nominal top elevation tolerance will account for upper element elevation tolerance
    v. High elevation tolerance of lower element only
        1. Low elevation tolerance of lower element will create larger joints, there is not much concerned with joints which are too large; therefore only high elevation tolerance of lower element is considered.

Step 2: Determine Tolerance Equations

a. Minimum Joint Tolerance (-) & Maximum Joint Tolerance (+)
    i. Minimum joint tolerance is more conservative in all situations; therefore (+/-) minimum joint tolerance
    ii. All tolerances discussed above applies to minimum joint width tolerance
    iii. Variation from specified end squareness/skew and local smoothness of any surface fabrication tolerances do not apply to maximum joint width. These do not apply to maximum joint width because these tolerance does not run the full height of element. The section of the element which is not affected by the tolerance will control the joint width or thickness.

Determine Maximum and Minimum Joint Width from the Following Possible Outcomes Assuming 100% Possibility of All Tolerances

a. Compared fabrication tolerances only vs. with erection tolerances with assumptions discussed above applied.
    i. Also looked into other tolerance assumptions to see how tolerances and risk factors were affected. These include;
        1. all fabrication and erection tolerances additive
        2. all fabrication tolerances additive
        3. fabrication length/height tolerance only
        4. fabrication length/height with erection tolerances additive
    ii. Concluded for vertical joints – half length fabrication tolerance + maximum of end skew/squareness and smoothness + half erection tolerance.
    iii. Concluded for horizontal joints – upper element fabrication height/depth tolerance + end skew/squareness fabrication tolerance + smoothness fabrication tolerance + lower element elevation tolerance.

Determine Joint Tolerance

a. Tolerance factor was determined by use of Risk Analyzer, an excel add-in.
    i. Risk Analyzer works by using Monte Carlo simulations. The Monte Carlo simulation is simply the creation of many "what if" cases to determine the expected results of a study. It allows users to account for risk in quantitative analysis and decision making.
    ii. A Monte Carlo Simulator will be used to determine tolerances with probability factored in. The tolerance factor, α, will then be used to get a more realistic tolerance value
b. Risk Analyzer input
    i. Normal distribution with zero mean
    ii. Standard deviation, σ, equal to half the element tolerance is used to determine the output standard deviation
    iii. Label input cells, maximum tolerance – fabrication length, fabrication end skew, fabrication smoothness, and erection/elevation tolerances
    iv. Label output cells, joint width tolerances, and associated equation referencing the input cells.
c. Risk Analyzer output yields standard deviation of a normal distribution
    i. Standard deviation of the joint tolerance is the output which is half the joint tolerance with the probability of occurrence factored in.
    ii. Two standard deviations to the left and right of the mean encompasses 95% of the allowable tolerances that will occur and the remaining 5% of the joints are considered a rejection. Rejection does not indicate that the pieces must not be used but the Engineer must determine if the joint is acceptable based on adjacent joints.
d. Joint tolerance factor, α
    i. Modified tolerance divided by maximum tolerance = α

Final Tolerance Equations and Probability Factors

a. Maximum tolerance is the joint tolerance of the additive fabrication and erection tolerances without probability of occurrence.

b. Modified tolerance is the joint tolerance of the additive fabrication and erection tolerances with the probability of occurrence factored in, two times the Risk Analyzer output standard deviation.
c. Choose factor which yields more conservative joint width tolerances between maximum and minimum
    i. Minimum joint tolerance is more conservative in all situations; therefore (+/-) minimum joint tolerance shall be used.

Results

The following pages contain the summary results of the Monte Carlo Simulations based on the process described above. Each page contains the results for a specific joint. The provision listed in the top right corner of each page corresponds to a Figure in the final guideline.