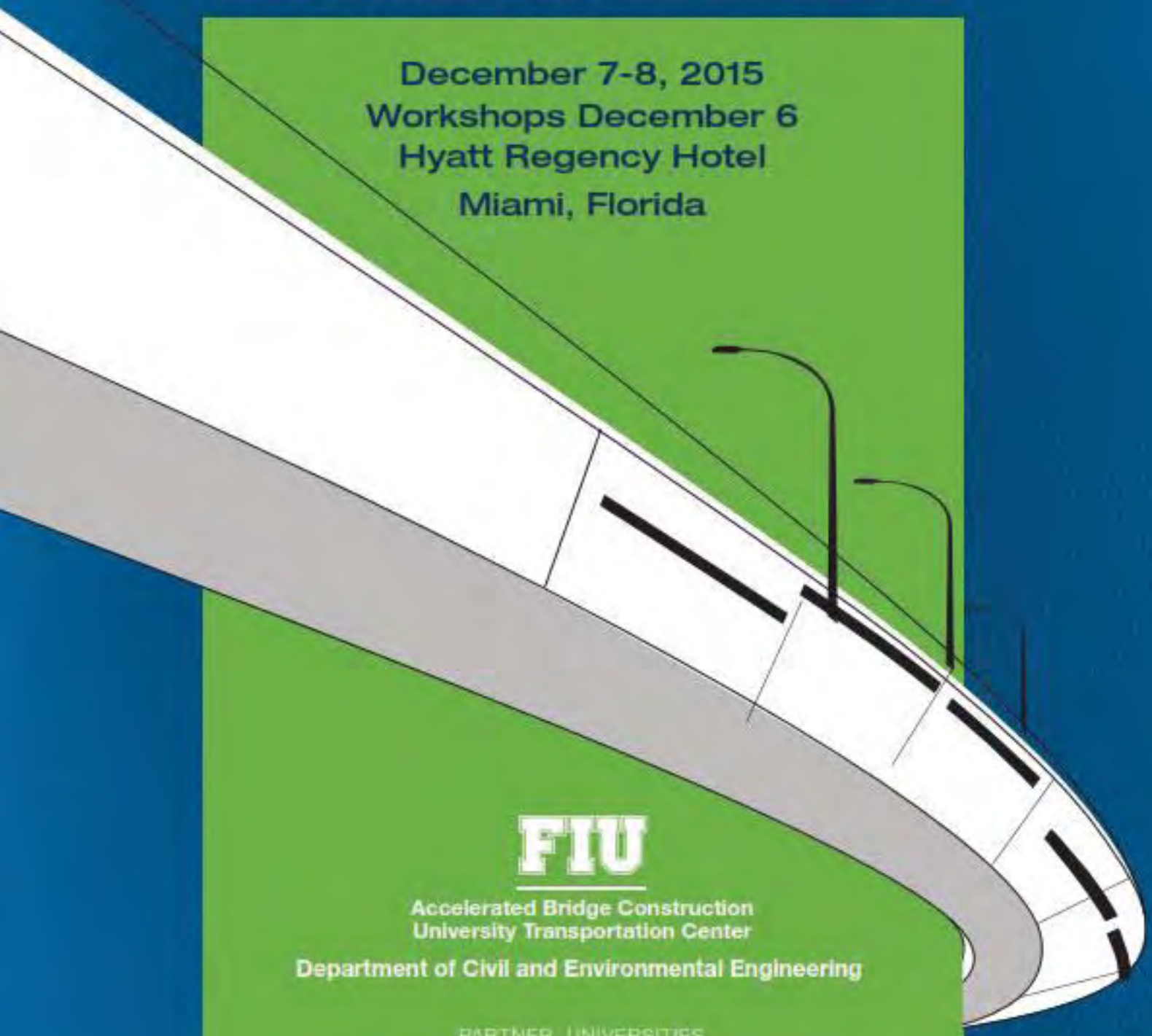
COLLECTION OF PAPERS AND EXTENDED ABSTRACTS

2015
NATIONAL ACCELERATED BRIDGE CONSTRUCTION CONFERENCE



December 7-8, 2015
Workshops December 6
Hyatt Regency Hotel
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Accelerated Bridge Construction University Transportation Center

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FOREWORD

Dear friends:

Welcome to Miami! We'd like to extend a warm welcome to attendees of the 2015 National Accelerated Bridge Construction Conference presented by the Accelerated Bridge Construction University Transportation Center at Florida International University (ABC-UTC) and FIU's Department of Civil and Environmental Engineering at College of Engineering and Computing.

Together with our partner universities, Iowa State University and University of Nevada-Reno, our objective at ABC-UTC is to advance the frontier of Accelerated Bridge Construction (ABC); develop new ABC knowledge; effectively transfer state-of-the-art ABC knowledge to the profession; develop a next-generation ABC workforce; and collaborate with the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), Department of Transportation (DOTs), other UTCs and the transportation profession to make ABC the best solution for the nation's aging bridge infrastructure in line with U.S. DOT's strategic focus on State of Good Repair.

This is our second annual conference being held to share with you the most current knowledge and technologies related to ABC. We have lined up an extensive schedule of workshops and sessions as well as several opportunities for networking with industry peers.

We hope you find the conference and workshop worthwhile and we encourage you to share with us your feedback and experiences.

Best wishes

Atorod Azizinamini, Ph.D., P.E.

Conference Chairperson

Professor and Chair

Director, Accelerated Bridge Construction – University Transportation Center (ABC-UTC)

Civil and Environmental Engineering Department

College of Engineering and Computing

Florida International University, Miami, Florida

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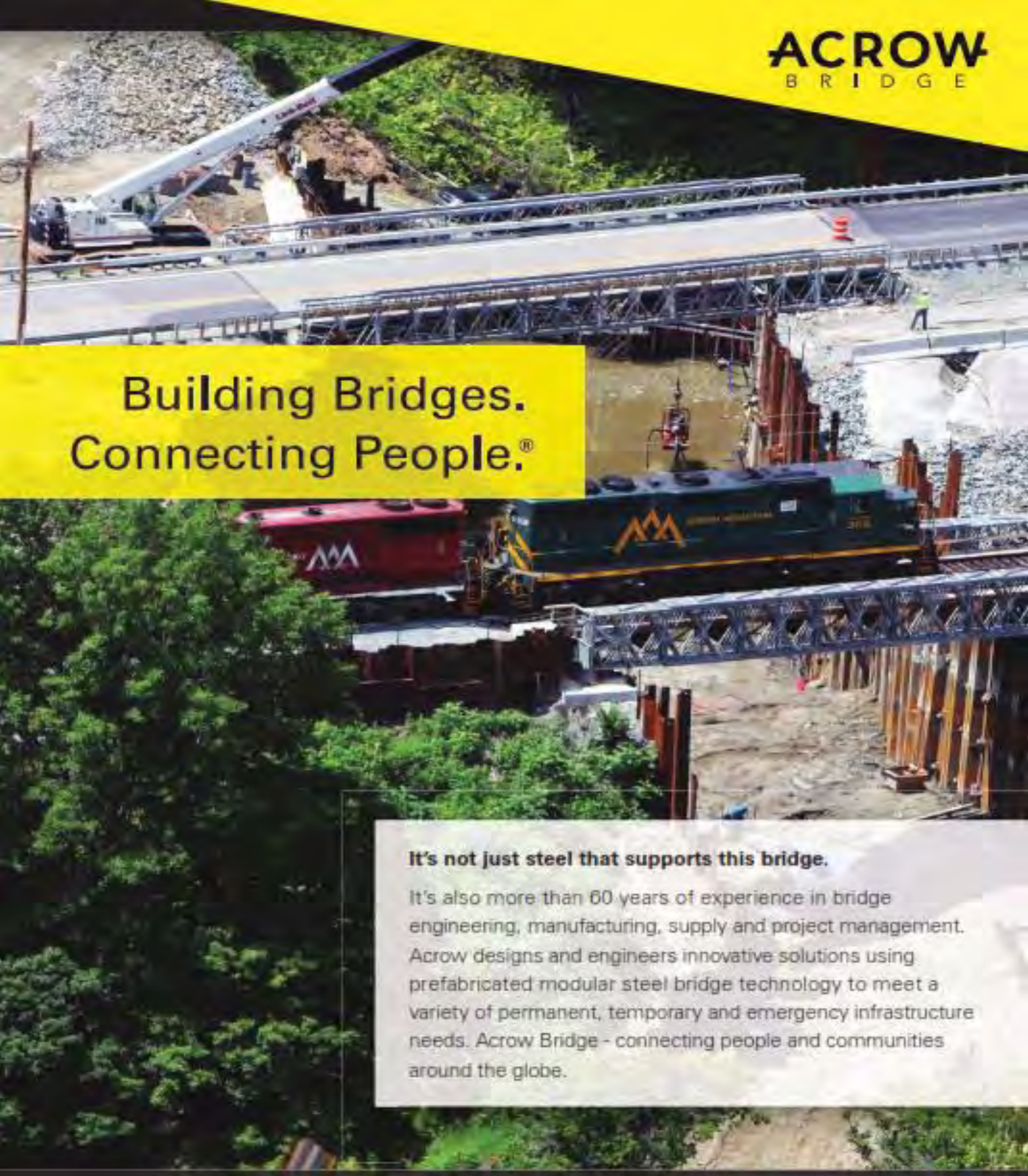
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PRE-CONFERENCE WORKSHOPS

A series of workshops were held one day before the conference on Wednesday, December 6, 2015.

WORKSHOP #1: EMERGING TECHNOLOGIES WORKSHOP

Sunday, December 6, 2015, 8:00 a.m.- 11:30 a.m.; 1:00 p.m. - 5:00 p.m.

Incorporating innovations into bridge projects leads to more successful projects. The driving force behind many innovations is an inventor who develops a product that provides improved performance over conventional practice. This workshop describes resources available to assist owners in incorporating emerging technologies into their projects, and describes how a State DOT was successful in incorporating them into its bridge projects. Technical details and benefits of several prominent proprietary products will then be discussed. The workshop will end with an open discussion of next steps to further advance the incorporation of emerging technologies into bridge projects..

Speakers:
John Huyer, P.E., Federal Highway Administration;
Wayne Frankhauser, Jr., P.E., Maine DOT;
John R. Hillman, P.E., HCB, Inc.;
Kay Jimison, SPS North America;
Matthew J. Macey, P.E., CDR Bridge Systems;
Rolando Moreau, P.E., Intelligent Engineering;
Eugene Sobecki, Acrow Corporation of America;
Kenneth L. Sweeney, P.E., AIT Bridges.

WORKSHOP #2: BRIDGE SYSTEM MOVES

Sunday, December 6, 2015, 8:00 a.m. - noon

Removing and installing entire spans is becoming an increasingly popular bridge replacement solution as bridge owners work to keep traffic moving while upgrading substandard bridges. Bridge moves with self-propelled modular transporters (SPMTs) and lateral slides using hydraulic jacks are the two most frequently used system move techniques. In this workshop, heavy move industry expert Sarens Group describe technical details of SPMT, lateral slide and alternative case studies they have engineered to minimize traffic impacts, reduce project cost and optimize construction cost.

Speakers:
Steven Sarens, Director Houston Operations, Sarens Group;
Mattias Price, Manager Technical Solutions Group, Sarens Group

WORKSHOP #3: PROGRAMMATIC IMPLEMENTATION OF ABC IN STATE DOTS

Sunday, December 6, 2015, 8:00 a.m. - noon; 1:00 p.m. - 5:00 p.m.

Implementation of ABC is more effective when the decision to do so is made from a programmatic perspective. A number of State DOTs have put together an ABC program; established ABC guidelines, policies, and internal and external processes; and have adopted bridge standards and details to include a more prefabricated theme to meet the objectives of ABC. This workshop described how these activities were accomplished in several DOTs, how it is working, challenges and opportunities for improvement, and the impacts of ABC to their bridge program in terms of efficiency, costs, and stakeholder feedback.

Speakers:
Ben Beerman, P.E., Federal Highway Administration;
Ahmad Abu-Hawash, P.E., Iowa DOT;
Thomas Donald, P.E., Massachusetts DOT;
Bruce Johnson, P.E., Oregon DOT;
David Juntunen, P.E., Michigan DOT;
Bijan Khaleghi, Ph.D., P.E.;
Dorie Mellon, P.E., California DOT;
William Oliva, P.E., Wisconsin DOT;
Mathew Royce, P.E., New York State DOT;
Carmen Swanwick, P.E., Utah DOT;
Wayne Symonds, P.E., Vermont Agency of Transportation

WORKSHOP #4: LIGHTWEIGHT CONCRETE – A TOOL FOR ACCELERATED BRIDGE CONSTRUCTION

Sunday, December 6, 2015, 8:00 a.m. - noon

To introduce owners, designers and contractors to the potential advantages of lightweight concrete (LWC) for accelerated bridge construction, including durability; to present examples of projects where lightweight concrete has been or could have been used; to equip designers with information needed for the practical implementation of lightweight concrete for ABC projects, including an introduction to the recent changes related to lightweight concrete in the AASHTO LRFD Bridge Design Specifications that simplify design and significantly improve the economy of lightweight concrete for some elements; and to introduce designers to the concepts of internal curing of conventional concrete mixtures by replacing a portion of the sand in a mixture with prewetted lightweight aggregate and the use of lightweight aggregate for geotechnical fill.

Speaker*:*
Reid W. Castrodale, President,
Castrodale Engineering Consultants, Concord, NC.

WORKSHOP #5: DESIGN CRITERIA AND CONNECTIONS FOR APPLICATION OF ABC IN SEISMIC REGIONS

Sunday, December 6, 2015, 1:00 p.m. - 5:00 p.m.

The objective of Accelerated Bridge Construction (ABC) in Seismic Regions workshop is to provide an opportunity for exchange of information and latest technology implementation in delivering ABC projects in seismic regions. Bridge substructure connections to superstructure and foundation are typically made at the beam-column and column-foundation interfaces to facilitate fabrication and transportation. In seismic regions, those interfaces represent locations of high moments, shears, and large inelastic cyclic strain reversals. Devising connections that can accommodate inelastic cyclic deformations and are readily constructible is focus of this workshop.

Speakers:
Atorod Azizinamini, Professor of Civil Engineering, Florida International University, Director, ABC-UTC;
M. Saiid Saiidi, Professor, Department of Civil & Environmental Engineering, University of Nevada, Reno. Director, CATBI, Principal Investigator, ABC-UTC-Seismic;
Max Stephens,
Dawn Lehman,
Charles Roeder, Professor of Civil Engineering, University of Washington.

WORKSHOP #6: STEEL BRIDGE DESIGN

Sunday, December 6, 2015, 8:00 a.m. - noon

To present up-to-date information for the design and construction of steel bridges using the latest design tools and technology and incorporating consideration for efficient, economical fabrication and construction.

Speakers:
Michael Culmo, P.E., CME Engineers, East Hartford, CT;
Ronnie Medlock, P.E., High Steel Structures, Lancaster, PA;
Robert Conner,Ph.D., Purdue University, West Lafayette, IN;
Chris Garrell, P.E., National Steel Bridge Alliance, Philadelphia, PA.

WORKSHOP #7: HPC: MATERIAL PROPERTIES, PRODUCTION, APPLICATION AND WHAT YOU SHOULD KNOW WHEN USING UHPC

Sunday, December 6, 2015, 8:00 a.m. - noon

The objective of this 8 hours long workshop is to provide detailed and essential information that bridge owners, designers and contractors' need to know when using or specifying the use of Ultra High Performance Concrete (UHPC). The workshop will provide the latest developments and information in the implementation of UHPC.

Speakers:
Paul White, P.E., P.Eng.Bridge Engineering Manager, UHPC/Ductal®, Lafarge North America;
Benjamin A. Graybeal, Ph.D., P.E. Structural Engineer, Office of Infrastructure Research Federal Highway Administration (FHWA);
Atorod Azizinamini, Ph.D., P.E., Professor and Chair, Director ABC-UTC at FIU

WORKSHOP #8: RE-USE OF SUBSTRUCTURES IN ABC PROJECTS

Sunday, December 6, 2015, 1:00 p.m. - 5:00 p.m.

Re-use of existing substructures is becoming an increasingly popular option as bridge owners work to address substandard bridges in their inventories. Eliminating the need to demolish existing substructures and construct new substructures saves time and money on bridge replacement projects. This workshop will describe how two States are re-using their bridge substructures. It will also describe two technologies – lightweight concrete and corrosion protection – that can assist in the re-use of substructures.

Speakers:
Jerry DiMaggio, P.E., Applied Research Associates, Inc.;
Reid W. Castrodale, Ph.D., P.E., Castrodale Engineering Consultants, PC;
Bruce E. Peterson, P.E., Modjeski and Masters; Maury Tayarani, P.E.,
Pennoni Associates Inc; David Whitmore, P.E., Vector Corrosion Technologies.

WORKSHOP #9: ABC'S OF ABC- ESSENTIAL ABC ELEMENTS FOR BRIDGE PROFESSIONALS

Sunday, December 6, 2015, 8:00 a.m. - noon

The workshop will introduce engineers, owners and contractors to the basics of ABC, introduced in a way that focuses on the key differences between ABC and conventional bridge design. The aim is to allow engineers familiar with traditional bridge design learn the key concepts of ABC in a few hours and advance the application of ABC to bridges.

Speakers:
Bala Sivakumar, Vice President, HNTB Corp. New York, NY

WORKSHOP #10: CONCRETE BRIDGE DESIGN WORKSHOP

Sunday, December 6, 2015, 1:00 p.m. - 5:00 p.m.

This series of talks will focus on key elements of precast, prestressed concrete bridge design and construction for ABC solutions. With more prefabrication, design and jobsite project personnel should take advantage of the flexibility that can be detailed into the elements as well as specific techniques used during the erection of precast elements. This workshop will provide the information on how to organize and lay out a jobsite for successful and safe erection of the structure.

Spekers:
Kevin Eisenbeis, P.E., S.E., Burns and McDonnell, Kansas City, MO;
Edwin McDougle, PE (retired), Ross Bryan Associates, Nashville, TN and e.Construct, Omaha, NE;
William Nickas, PE, Managing Director, Transportation Systems for Precast/Prestressed Concrete Institute (PCI), Chicago, IL

CONFERENCE PROGRAM

CONFERENCE PRESENTATIONS- MONDAY DECEMBER 7					
	8:00 AM - 10:00AM **GENERAL SESSION**				
	G-1: STATE ABC EXPERIENCES I	**G-4: LATERAL SLIDE CASE STUDIES I**	**G-6: RE-USE OF EXISTING SUBSTRUCTURES FOR ABC PROJECTS**	**G-11: ABC RESEARCH I**	**G-9: ABC CONSTRUCTION SOLUTIONS**
10:30 AM – 11:00 AM	**Hartford Vermont Bridge Replacement - SIBC and CMGC Project Delivery** *Kristin Higgins, Vermont Agency of Transportation*	**THE LARDO BRIDGE SLIDE** *Brian Byrne, Lochner*	**Foundation Characterization and Reuse in ABC Projects** *Frank Jalinoos, FHWA*	**Slide-in bridge construction cost estimation tool** *AJ Yates, Michael Baker International*	**Mechanized ABC of Large-Scale Projects** *Marco Rosignoli, Parsons Brinckerhoff*
11:00 AM – 11:30 AM	**Fort goff creek bridge fish passage.** *Mario Guadamuz, California Department of Transportation (CALTRANS)*	**M-100 Superstructure Slide in the City of Potterville, Michigan** *Daniel Broekhuizen, AECOM*	**Load Rating of Bridge Foundations** *Anil Agrawal, The City College of New York*	**Precast bridges subjected to very important horizontal loads** *Luis Javier Sanz, Imagina-Titandol SAS*	**Is that Bridge Open Yet? ABC lessons from a Contractor's perspective** *Michael Zicko, HCB, Inc.*
11:30 AM – 12:00 PM	**The Value of ABC: A Rural DOT Perspective-** *Jennifer Fitch, Vermont Agency of Transportation*	**Lateral Slide of Historic Bridge in Washington State** *Kevin Dusenberry, Jacobs*	**Assessement, Risk Management And Rehabilitation Of Existing Structural Foundations** *Jerry DiMaggio, Applied Research Associates*	**Effect of the interface moisture content on the bond performance between a concrete substrate and a non-shrink cement-based grout** *Igor De la Varga, SES Group*	**Innovative Construction Methods to Replace the division Street Bridge over the North Branch Canal in Chicago, Illinois** *Eugene Sobecki, Scott Patterson, Acrow Bridges*

CONFERENCE PRESENTATIONS- MONDAY DECEMBER 7					
	G-3: EVALUATION AND MONITORING OF ABC BRIDGE MOVES	**G-4: LATERAL SLIDE CASE STUDIES II**	**G-10: PBES CONNECTION DETAILS AND MATERIALS**	**G-11: ABC RESEARCH II**	**G-18: ADVANCED MATERIALS FOR ABC**
1:00 PM – 1:30 PM	**Superstructure monitoring during SPMT movement** *Ryan Drefus, Geocomp Corporation*	**Highway 406 Glendale Avenue Overpasses Rehabilitation: Lateral Slide Rapid Bridge Replacement** *David Cerullo, MMM Group Limited*	**Evaluation and Design of Resilient Concrete Filled Tube Column-to-Precast Cap Beam Connections** *Max Stephens, University of Washington*	**SHRP2 ABC Details Lead to a Successful Bridge Replacement in Maine** *Tom Kendrick, McFarland Johnson, Inc.*	**Understanding the applications and Advantages of Calcium Sulfo-Aluminate (CSA) Cement Technology in Accelerated Bridge Construction** *Chris Davis, CTS Cement Mfg. Corp / Rapid Set Products*
1:30 PM – 2:00 PM	**Friction Values for Slide-In Bridge Construction** *Kristopher Johnson, Utah State University*	**Going the Distance: The Lateral Sliding of Four Railway Bridges at West Toronto Diamond in Toronto.** *Brent Archibald, Parsons*	**Rapid Bridge Deck Joint Repair and Rehabilitation** *James Nelson, Iowa DOT*	**Development of prediction model for ultimate flexural and shear capacities of jointed precast deck panels in bulb-tee girder system** *Imad Eldin Khalafalla, Ryerson University*	**Recent Changes in AASHTO LRFD Specifications Regarding Lightweight Concrete** *Gary Greene, Trine University*
2:00 PM – 2:30 PM	**Structural Response Monitoring to Assess Bridge Condition During Construction** *Chris Kavars, SENSR LLC*	**Larpenteur avenue bridge** *Logan Julander, Michael Baker International*	**Precast Substructures for ABC— Case Studies in Design & Construction** *Alan Phipps, Figg Bridge Engineers*	**Data Base of all available ABC projects and research information** *David Garber , FIU*	**Lightweight Aggregate as Geotechnical Fill for ABC Projects** *Reid Castrodale, Castrodale Engineering Consultants, PC*
2:30 PM – 3:00 PM	**Bridge Pier Movement and Forces during Slide-In** *Upul Attanayake, Western Michigan University*		**FIELD-CAST CONNECTIONS FOR PREFABRICATED DECK PANELS: Performance of Pre-Bagged Connection Grouts** *Zachary Haber, Professional Service Industries, Inc.*	**Extending application of Simple for Dead Load and Continuous for Live Load (SDCL) steel bridge system to high seismic areas** *Ramin Taghinezhad , FIU*	**Use of Lightweight Concrete for Bridges Moved into Place** *Reid Castrodale, Castrodale Engineering Consultants, PC*

CONFERENCE PRESENTATIONS - TUESDAY DECEMBER 8					
	G-1: STATE ABC EXPERIENCES III	**G-7: ADVANCING SEISMIC RESEARCH FOR ABC I**	**G-13: ABC CASE STUDIES I**	**G-17: PROMOTING ABC**	**G-19: MISCELLANEOUS I**
8:00 AM – 8:30 AM	**New Development in Accelerated Bridge and Culvert Construction in Washington State** *Bijan Khaleghi, WSDOT*	**A Class of V-Connectors for Deck-Pier or Pier-Footing Joints with Combined Advantages of Integrated Design and Seismic Isolation while Enabling ABC** *Su Hao, ACII, INC.*	**ABC of bridge no. 465 Rhode island i-195 ramp (dr-2) over warren avenue** *Aboud J Alzaim, Louis Berger*	**Latest development in steel bridge design and construction with focus on ABC** *Bill McEleney, NSBA*	**The Future of Bridge Design** *Barbara Day, Bentley Systems*
8:30 AM – 9:00 AM	**Evolution of Design Details for Iowa DOT Modular Bridge Construction: Case Study of Little Silver Creek Accelerated Bridge Construction Project** *Curtis Carter, Iowa DOT*	**Grouted Splice Sleeve Connections for Bridge Piers in Seismic Regions: Experiments and Analysis** *M.J. Ameli, University of Utah*	**12 Day Bridge Replacement Project in Vermont with a 47 Mile Detour** *David Kull, McFarland Johnson, Inc.*	**Latest development in concrete bridge design and construction with focus on ABC** *William Nickas, PCI*	**Accelerated Bridge Construction - Huey P. Long Bridge Truss Lift Monitoring to Mitigate Risk** *Thomas Weinmann, Geocomp Consulting*
9:00 AM – 9:30 AM	**TDOT CM/GC Experience with ABC – Fast Fix 8 in Downtown Nashville, Tennessee** *Lia Obaid, TDOT*	**Seismic Repair of Damaged Precast RC Bridge Columns Connected with Grouted Splice Sleeves** *Chris Pantelides, University of Utah*	**Sellwood Bridge Diversion Alignment and Bridge Translation** *Scott Nettleton, T.Y. Lin International*	**Social Media for ABC: State of Practice and Workshop for ABC Project Owners** *Eliza Partington, Figg Bridge Engineers*	**Decision Criteria for Deploying Accelerated Bridge Construction** Gordana Herning, Rutgers University Center for *Advanced Infrastructure and Transportation*
9:30 AM – 10:00 AM	**Accelerated Bridge Construction in Alaska** Leslie Daugherty, Alaska Department of *Transportation & Public Facilities*	**Innovative ABC columns with copper-based SMA and ECC for seismically active regions** *Sebastian Varela, University of Nevada, Reno*	**Bent cap design for IH-635 Managed Lanes Project** *Ignacio Navarro, Ferrovial Agroman US Corp*	**ABC national update and overview** *Ben Beerman, FHWA*	**Robotic Construction in ABC Projects** *Alireza ValiKhani , FIU*

CONFERENCE PRESENTATIONS - TUESDAY DECEMBER 8					
	G-2: ABC PROJECTS WITH GRS-IBC	**G-8: ABC PROJECTS USING SPMT**	**G-13: ABC CASE STUDIES II**	**G-11: ABC RESEARCH IV**	**G-19: MISCELLANEOUS II**
10:30 AM – 11:00 AM	**Phased Accelerated Bridge Construction with GRS/IBS Abutments** *Ralph Verrastro, Bridging Solutions, LLC*	**SPMT for Accelerated Bridge Construction – High-grove Underpass** *Andy Quach, Caltrans*	**Reconstruction of the Willis Avenue Bridge with Implementation of ABC Technology** *Wei Wang, UrbanTech Consulting Engineering*	**Innovative ABC solutions using UHPC** *Mohamadreza Shafieifar , FIU*	**200+ Years of Accelerated Bridge Construction in America** *John Hillman, HCB, Inc.*
11:00 AM – 11:30 AM	**First-Ever Innovative Application of GRS-IBS in the Garden State** *Ahmad Faqiri, Pennoni Associates Inc.*	**The 123's of SPMT's for ABC** *Charles Neth, Siefert Associates, LLC*	**NJ Route 18 Bridge over US Route 1 Superstructure Replacement and Widening utilizing Prefabricated Superstructure Units** *Gregory Ricks, HNTB Corporation*	**ABC Connections with UHPC** *Benjamin Graybeal, FHWA*	**Seismic behavior of precast hollow-core columns for accelerated bridge construction** *Mohamed ElGawady, Missouri University of Science and Technology*
11:30 AM – 12:00 PM	**Implementation and Research Case Study on a GRS-IBS Project** *Bob Gilbert, Bartlett & West*	**Rehabilitation of the Park Avenue and Watchung Avenue Bridge for New Jersey Transit utilizing Self-Propelled Modular Transporters** *David Mykulak, HNTB Corporation*	**Salmon river bridges** *Lee Wegner, Hanson Structural Precast*	**Construction of UHPC Connections** *Benjamin Graybeal, FHWA*	**Safety Indices(ßvalues) for a full depth Precast Concrete Deck Panel connected with Accelerated Bridge Construction (ABC)** *Chandrasekhar Putcha, California State University, Fullerton*

CONFERENCE PRESENTATIONS - TUESDAY DECEMBER 8					
	G-1: STATE ABC EXPERIENCES IV	**G-7: ADVANCING SEISMIC RESEARCH FOR ABC II**	**G-5: ABC SUBSTRUCTURES SOLUTIONS**	**G-16: DESIGN OF ABC PROJECTS FOR SERVICE LIFE**	**G-12: ADVANCEMENT OF PREFABRICATED BRIDGE ELEMENTS**
1:30 PM – 2:00 PM	**Connecticut Department of Transportation State Project No. 131-194/195 I-84 over Marion Avenue, Southington Bridge Superstructure Replacement using Self-Propelled Modular Transporters (SPMTS)** *Mary Baker, Connecticut DOT*	**Seismic Accelerated Bridge Construction with Low-Damage Segmental Bridge Columns incorporating Damage-Resistant Joints** *Mohammad T. Nikoukalam, University of Colorado - Boulder*	**Experimental Study on Precast Segmental Bridge Columns with Semi-rigid Connections** *Yu-Chi Sung, National Taipei University of Technology, Department of Civil Engineering*	**Galvanizing your Bridge for Corrosion Protection** *Kevin Irving, AZZ Galvanizing Services*	**Bridge in a Box** *Kenneth Sweeney, AIT Bridge Systems*
2:00 PM – 2:30 PM	**FDOT Precast Bent Cap Development and Implementation** *Steven Nolan, FDOT*	**Seismic Evaluation of a Precast PT/UHPC Bridge Column with Pocket Connection and Precast Footing** *Alireza Mohebbi, University of Nevada, Reno*	**Reconstruction of the First Single Point Urban Interchange Bridge In Michigan Utilizing Prefabricated Substructure Elements** *Ihab Darwish, alfred benesch & Co.*	**Modern non-bituminous flexible plug expansion joints – minimizing noise, maximizing driver comfort and accelerating bridge maintenance** *Gianni Moor, Mageba USA*	**Experimental study of A Precast Barrier Wall System performance for Bridge Decks** *Zhangzhen Wei, Department of Bridge Engineering, Tongji University, ShangHai*
2:30 PM – 3:00 PM	**Wisconsin DOT's Development of Standardized and Interchangeable PBES - Piers** *William Oliva, Wisconsin DOT*	**The seismic ductility device** *D. Stephen Fitz, Structural Component Systems, Inc.*	**Precast Pier Construction on Northwest Corridor** *Ali Ghalib, Parsons*	**Service Life Extension of Existing Bridge Substructures** *Jason Chodachek, Vector Corrosion Technologies, Inc*	**Construction of the Second Generation of Precast Concrete Deck System NUDECK** *Fouad Jaber, Nebraska Department of Roads*
3:00 PM – 3:30 PM	**ABC in Alabama** *Paul Froede,, Alabama Deptartment of Transportation*	**Shaking Table Performance of a New Precast Bridge Bent System with Pre-Tensioned, Rocking Columns** *Travis Thonstad, Z*	**Post-Tensioned Precast Pier Caps for the Honolulu Rapid Transit Project** *Joseph Krajewski, HNTB*	**Experimental testing of UHPC joints of precast concrete bridge deck** *David Citek, CTU in Prague - Klokner Institut*	**ABC Construction with an Innovative and Cost-Effective Full Depth Precast Deck Solution** *Eddie He, AccelBridge*
3:30 PM – 4:00 PM	**Utah's Experiences with ABC** *Carmen Swanwick , Utah Department of Transportation*	**Caltrans ABC Research program** *Tom Ostrom, Caltrans*	**Substructure Considerations for Successful Accelerated Bridge Replacement Projects** *David Whitmore, Vector Corrosion Technologies*	**Customized Manual for Design of ABC Projects for Service Life** *Azadeh Jaberi , Morgan Dickinson , Atorod Azizinamini , FIU*	**Sustainable precast concrete segmental bridges achieved using ABC** *Linda Figg, Figg Bridge Engineers*

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G-1: STATE ABC EXPERIENCES

FORT GOFF CREEK BRIDGE

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EXTENDED ABSTRACT

The Fort Goff Creek Bridge is a streambed restoration project located in a remote part of Northern California along the Klamath River on Route 96 in Siskiyou County. The project replaced a 15 foot diameter steel pipe culvert with a 60 foot single span precast structure. The culvert posed a barrier to fish movement upstream. Construction of the bridge allowed for the channel section and the stream bed beneath the highway to be restored to a more natural state providing unimpaired passage for anadromous fish.

The project presented a number of challenges to conventional cast-in-place construction. The site is located in a severe climate area where freeze thaw cycles and heavy salting occur frequently and chain use is common. These conditions require special attention to the longevity of the deck and the structure in general. The nearest batch plant is located approximately 90 minutes away, which created a situation where cast-in-place concrete quality could be easily compromised by traffic delays. Traffic and environmental constraints called for one season construction and minimized impacts to the stream bed and surrounding areas.

Multiple structure types were evaluated during the planning phase of the project. A prefabricated bridge element system (PBES) was identified as the preferred construction method for the 36 foot wide, 60 foot long single span structure. The drivers for accelerated bridge construction (ABC)/PBES included improved quality of concrete elements, reduced environmental impacts, and the single season restriction.

The PBES structure design utilized precast voided slabs, precast abutments, precast wing walls, and prefabricated steel barrier rail. Concerns over constructability issues, associated schedule delays and other challenges of implementing innovative methods were addressed through the use of the ABC Toolkit (1) and funding from the Strategic Highway Research Program 2 (SHRP2) Implementation Assistance Funds for Innovative Bridge Designs for Rapid Renewal. The ABC Toolkit, a SHRP2 product, “provides a series of design and construction concepts for prefabricated elements and their connections” (1).

The construction contract was awarded in the spring of 2014 by the design-bid-build/low bidder method and detour construction began on May 30th, 2014. Single lane signalized traffic was carried on a detour over a temporary culvert for the duration of the project. Precast abutments were founded on a single row of 30 inch diameter cast-in-drilled-hole (CIDH) piles with permanent casing. Due to difficulties with drilling and drilling equipment the pile installation was completed in early September. The precast seat type abutments were erected in segments to keep element weights below 100 kips for ease of transportation and placement. Large voids formed with corrugated metal pipe fit over the extended pile cage and were filled with 6ksi grout. A 275 ton crane was used by the contractor to place the abutment elements in 2 days. The abutment elements were then connected by grouted keyways and post tensioned tie rods. Once the abutment grout had reached strength, the precast voided slabs and wing walls were set in a single day. Precast wing walls were connected to the abutment by closure pours and the voided slabs were connected to each other by grouted keyways and post tensioned tie rods. A riding surface of polyester concrete overlay was placed on the precast superstructure in mid-October and then prefabricated metal barrier rail was bolted to the structure by way of anchorages embedded in the precast curb of the exterior girders. Roadway work was completed and the traffic was shifted to the permanent structure on November 12th, 2014.

Best practices from the ABC Toolkit incorporated into the Fort Goff Project include the use of a single row of piles under precast abutment, repeatable elements, pick weights under 100 kips, pre-assembly of substructure elements prior to shipping and the incorporation of fabrication and erection tolerances in the plans and special provisions. Other items added for ease of implementation were the use of prefabricated rail, including the rail curb in the precast exterior slab elements, providing extra overlay thickness to accommodate vertical tolerances and calling out a construction sequence on the plans.

An unexpected benefit of PBES construction was the mitigation of a 63 day schedule overrun due to foundation construction problems. Pile installation poses a significant schedule risk on any project, especially under conditions of wet and difficult drilling. The schedule delays introduced by the CIDH piling on the Fort Goff Creek Bridge were effectively offset by the rapid assembly of the PBES structure (71 days for foundation construction versus 23 days for bridge construction). The use of precast elements removed the threat of the project extending into a second season, thereby avoiding addition cost, extended traffic delays, and significant environmental impacts.

The Fort Goff Creek Bridge project provided several lessons learned that will allow Caltrans to improve ABC/PBES project delivery going forward. Engineers recognized several opportunities to improve the shop plan review process including a longer delayed start to provide adequate time to review the

increased number of shop plans and concurrent shop plan submittal to identify conflicts between prefabricated components. Going forward, Caltrans is considering the requirement of building information modelling (BIM) of precast elements and connections to avoid congestion and improve constructability. BIM would be particularly helpful identifying geometric complications introduced by skews, cross slopes, and horizontal and vertical curves.

ABC/PBES construction successfully delivered a one season solution for the Fort Goff Creek Bridge project. This type of construction was well received by the contractor and Caltrans construction staff. Program managers in Northern California where the bridge was built have identified several future projects that would benefit from a similar approach and envision widespread application in fish passage projects statewide. Caltrans continues to pursue ABC and PBES on a larger scale in order to effectively mainstream ABC in California.

Figure 1: Fort Goff Creek Bridge

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1. *Transportation Research Board (2013) Innovative Bridge Designs for Rapid Renewal ABC Toolkit, SHRP 2 Report S2-R04-RR-2.*

THE VALUE OF ABC: A RURAL DOT PERSPECTIVE

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ABSTRACT

This paper provides an overview of the impedance for the ABP, implementation strategies with internal and external customers and general approach to ABC with an emphasis on value of ABC and recommendations on how these benefits could be measured throughout the organization and on a program and project level. Case studies will also be included to provide supporting documentation.

INTRODUCTION

The value and benefits of accelerated bridge construction (ABC) are often easier to distinguish and realize in urban environments characterized by high traffic volumes, congestion and greater roadway densities. In these areas, ABC dramatically reduces work zone road user costs as compared to other maintenance of traffic options including lane closures, temporary bridges and phased construction. This coupled with other project costs including right-of-way acquisition, project administration, environmental permitting and utility relocation, or lack thereof, often results in lower project costs as compared to conventional bridge construction making it an easy sell to lawmakers, the public and other stakeholders. Moreover at a time when many of our urban highways are over capacity and customers have come to expect a reliable transportation network, building and maintaining support for efficiently preserving our highway assets through the use of ABC and short term roads closures can be easier especially when detour routes are relatively short adding only minutes to a standard commute. However, in rural states like Vermont, it can be harder to justify the value ABC where average daily traffic volumes and associated roadway user costs tend to be low leading to higher construction costs as compared to conventional construction. In addition, roadway networks are scarce resulting in lengthy detours making it difficult to garner public support.

So what is the value of ABC and how should it be measured especially if construction costs are equal to or greater than conventional construction? Like other states, many of the typical benefits hold true regardless of geography. For example, by closing a road rather than installing a temporary bridge, impacts to right-of-way, environmental resources and utilities are reduced or, in some cases, eliminated altogether, facilitating expedited project delivery and rapid replacement of deteriorating infrastructure. With Vermont's lengthy and arduous environmental permitting and right-of-way acquisition (ROW) processes,

the use of ABC has substantially reduced the time it takes to deliver bridge rehabilitation and replacement projects ultimately lowering design costs and reducing resource demands. Short term road closures also improve safety for motorists and construction workers alike by rerouting traffic around rather than through the work zone. Just last year (2014), a car traveling on one of Vermont's interstates struck and killed a construction worker. As our mission, like many other transportation agencies, includes providing for the safe movement of people and goods, the Structures Section recently adopted a general rule of considering road closures as preferred option for maintaining traffic unless it is deemed impractical during the project initiation phase. While not always obvious, especially in cases with lengthy detour routes, short term road closures also minimize impacts to the traveling public and commerce by significantly reducing onsite construction duration. Results from a public survey following three consecutive rapid bridge replacement projects on VT 73 showed that the majority of respondents (82%) felt very satisfied with ABC and short term road closures even though the detour length end to end was 51 miles over mountainous terrain.

In 2012, the Vermont Agency of Transportation (VTrans) created the Accelerated Bridge Program (ABP). Since its inception, VTrans has reliably expedited project delivery and reaped many of the common benefits described above which are often easy to measure. This success has added unanticipated value by becoming ingrained in our organizational culture promoting innovation throughout all phases of project delivery and other business practices. In addition, the ABP has gained significant support from local politicians and, with this support, has been able to pass legislation that further enables and promotes the program. Finally, we have found that ABC adds substantial value to legacy projects that were once shelved due to public opposition to conventional construction.

EXPEDITING PROJECT DELIVERY

At a time of increased federal funding associated with American Recovery and Reinvestment Act (ARRA) along with an aging bridge population in need of rehabilitation or replacement, the VTrans examined various strategies and mechanisms to expedite the delivery of bridge projects. However, given the narrow widths common to Vermont's workhorse bridges ruling out phased construction coupled with our lengthy and arduous ROW and environmental permitting processes needed to maintain traffic on temporary bridges, this goal proved difficult. In the past, ABC with short term road closures was only used on a case by case basis when all other alternatives were ruled out. However, VTrans quickly recognized that by using this innovative approach to delivering bridge projects, the project development process could be streamlined.

In 2012, the Structures Section was reorganized creating the Project Initiation and Innovation Team (PIIT) and ABP. Essentially a scoping unit, the PIIT was formed to ensure an efficient, consistent and programmatic approach to identifying the best alternative for rehabilitating and replacing deteriorated bridges and culverts. This process considers the needs of the bridge, maintenance of traffic options, construction practices and contracting methods along with an emphasis on the context of the corridor and community involvement. Rather than looking at ABC and short term closures as the last choice, these methods are examined alongside more conventional construction practices and only discarded if found impractical.

To assure the successful implementation of ABC on a statewide basis, the ABP was established to specialize in expedited project delivery using prefabricated elements and systems (PBES) and short term road closures. By minimizing project impacts, VTrans has been able to reduce the project development phase from 60 months for conventional construction projects down to just 24 months, or a 60% reduction, allowing the Structures Section to respond quickly to increases in funding, emergency bridge replacement projects and more stringent bridge inventory performance measures.

The ABP could not have come to fruition at a better time. In August 2011, Tropical Storm Irene (TSI) pummeled the slopes and valleys of Vermont with heavy rain and wind, severely damaging more than 500 miles of state roads and 200 bridges, isolating 13 communities (1). Shortly following the initial response to repairing Vermont's transportation network, the ABP tapped into lessons learned delivering all 15 Emergency Relief (ER) projects within 12 to 24 months. This would not have been possible with conventional construction methods and associated impacts. In addition, by minimizing the footprint of a project, resource demands have also been greatly reduced or eliminated not only saving time and money but also allowing for precious resources to be allocated to larger more complex projects. Something that is necessary at a time when state agencies are asked to do more with less.

CREATING A CULTURE THAT VALUES INNOVATION

Like most large organizations, it is often difficult to innovate because standard operating procedures and associated habits are hard to change. The same holds true for roadway network users. At the onset of the ABP, team members met with various stakeholders around the state, including internal resource groups, the public, legislators, regional planning commissions and contractors, to demonstrate the value of ABC and PBES and garner support for the program. These forums provided an opportunity to brainstorm ideas on how best to implement the program within the Agency and discuss any comments or concerns from the public, emergency responders and contractors. This early and continued collaboration created invaluable partnerships and a means for these stakeholders to become invested in the program.

In addition, Project managers (PMs) within the ABP were given a great deal of latitude and encouraged to explore and vet different strategies for streamlining the project delivery process. This allowed for creativity and calculated risk taking. Since most of the initial projects were related to TSI, the ABP had support at the highest levels within VTrans to meet or exceed the time requirements associated with ER funding along with heightened urgency within the design and resource groups to restore the transportation network. As these project progressed through the delivery process, members of the ABP team met regularly to share lessons learned and recommend strategies to incorporate into standard operating procedures. This included determining which development activities could occur concurrently, how to effectively and efficiently coordinate with the resource groups, best practices for public engagement and standardizing plan sets and specifications. By creating a more inclusive and collaborative process, teams working on ABP projects became invigorated and excited to take a fresh look at modifying standard business practices allowing them to take pride and ownership of what can be accomplished when inspired to innovate.

As the first projects went out into construction, communities were hesitant to the notion of short term road closures. Most residents, business owners and emergency services had never heard of ABC and were uncertain that a bridge could be replaced in such short timeframe. To alleviate these concerns, PMs worked closely with affected communities to provide real time information and assurance that short term road closures with PBES reduces traffic impacts and is a better approach to replacing bridge structures where practical. In addition, most contracts were and still are incentivized to open the road early or on time providing yet another motivation for the projects to be successful. As the first projects were completed and roads reopened to traffic, communities began to embrace ABC with many towns holding bridge opening celebrations on their own accord. With these initial victories, the ABP has gained momentum allowing the program to become ingrained in our organizational culture promoting innovation throughout all phases of project delivery and other business practices.

GARNERING POLITICAL CAPITOL

As word began to spread around the state about the ABP along with recent successes, Vermont legislators became increasing interested to learn more about rapid bridge replacement projects. Staff from the ABP were invited to provide an overview of the program to the House and Senate Transportation Committees. Members of both committees were impressed with how VTrans embraced the use of ABC on a programmatic level and all of the associated benefits. During these early meetings with Vermont legislators, each transportation committee brainstormed different ideas on how to promote rapid bridge replacements during short term roads closures on a statewide basis ultimately leading to the enactment of

Act 153 in 2012. Act 153 reduces the town share by 50% on town highway projects if the town elects to close the road rather than install a temporary bridge providing a considerable financial incentive for short term road closures. This legislation alone has helped propel the ABP forward. To date, 20 towns have elected to take advantage of Act 153. Not only has this helped promote the use of ABC on town highway projects but has also had a rippling effect on state projects as well. Once towns become accustomed to ABC with short term road closures as a means to deliver projects more efficiently with minimal impacts, they are more receptive to the idea on state highway projects.

Since 2012, the Vermont House and Senate Transportation Committees have asked for an annual updates on the ABP. Committee members are always eager to hear about advancements in the program, the number of towns that have taken advantage of Act 153, an overview of current projects under development in the ABP and the list of bridges that will be replaced the following construction season. Senior members of program also use this forum to showcase innovative projects using new ABC construction techniques to highlight that the Agency is committed to finding solutions to reduce onsite construction time on complex as well as standard bridge projects. This continued partnership and yearly check in with the transportation committees has been invaluable to the deployment of the ABP and ABC on a statewide basis. These meetings allow for an open dialog with lawmakers to promote the benefits of ABC, report out on current statistics and brainstorm other ideas to promote the program. In fact, in most cases, the legislators ask what they can do to endorse the program. By engaging the transportation committees and other legislators around the state, they in turn have become a partner in the program's success.

USING ABC TO DELIVER LEGACY PROJECTS

Several projects within the Structures Program had been on the books since the early 1980's. Impacts from traditional construction methods along with scope creep caused these projects to come to a standstill. Many of these projects were put on hold due to public opposition. Concerns were widespread including: 1) temporary and permanent impacts to recreational, environmental and archeological sensitive areas, 2) traffic impacts due to phasing and single lane temporary bridges especially in communities that rely heavily on travel and tourism for a healthy economy, 3) replacing the existing structure with one that addresses the context of the corridor, 4) accommodating bicyclists and pedestrian use, and 5) general approach to project delivery. Many communities felt that VTrans was dictating an engineering solution rather than engaging them to come to a collective resolution that not only met the needs of the deficient bridge structure but the other considerations noted above. Unfortunately, these projects were put on hold until a different solution was developed only to be met with more public opposition resulting in legacy

projects. This created significant inefficiencies in the project development process including lengthy schedule delays and increased design costs. The majority of these projects were shelved due to community concerns over project impacts and traffic management. With the establishment of the ABP in 2012, all of these legacy projects were reexamined to determine if ABC with short term closures was the right delivery approach and, as it turns out, it was the only approach that was met with public support from affected communities and other key stakeholders.

The Middlebury Sand Hill Bridge is a prime example of a legacy project. Located along VT 125 in East Middlebury, VT over the Middlebury River, the historic 49' long arch structure built in 1924 was programmed for replacement in 1983. Bridge 13 was programmed for replacement due to its poor condition. However, there were a number of other deficiencies including bridge and approach widths (24' 6" vs. 30'), bridge rail and banking as well as concerns over the suitability of the rock the bridge was founded on. In addition, the bridge was surrounded by old mill sites, a swimming hole identified as a recreational resource and was also designated as historic. All of these represented significant hurdles in the project development process.

The original scope included the installation of a temporary bridge on the upstream side of the existing structure in addition to widening the bridge to meet state standards. Due to its historic designation, the new structure also needed to have the same look and feel as the existing structure. Given the resources surrounding the structure, the public was adamantly opposed to any alternative that had significant impacts to valued recreational and archeological sensitive areas. Phased construction was not an option given the bridge type and inadequate bridge width. All of these factors caused the project to come to a halt. An innovative solution that minimized project impacts and met historic requirements almost seemed impossible until a short term road closure was considered. This traffic management strategy eliminated impacts from a temporary bridge structure and was much more palatable to the East Middlebury community. This got the project moving forward again. However, traditional arches can take at least one construction season to build, too long for a short term road closure. So engineers developed a design utilizing all prefabricated components to create an arch like structure, a NEXT beam bridge with a separate precast arch both founded on their own precast footings as shown in Figure 1. The sides of the arch were enclosed with precast panels creating the look of a solid concrete arch.

Ultimately, for many of these types of legacy projects, ABC with short term closures is the only viable solution. In the case of the Middlebury Sand Hill Bridge, designers vetted alternatives which always included a temporary bridge for over 28 years. However, once a short term road closure was considered, the project was delivered in just 3 years highlighting the importance of using innovation to remove

impediments to delivering projects. The new arch was constructed in 42 days, 3 days short of the allowable 45 day closure period.

Figure 1: Middlebury Sand Hill Construction

PARTNERING WITH LOCAL COMMUNITIES

Often times public involvement may be construed as an impediment to project delivery. The public and other customers often have differing opinions about bridge features, traffic calming measures, safety improvements, accommodating alternative modes of transportation and the best approach to maintaining traffic during construction. In some cases, developing consensus can be difficult. However, investing in public involvement at the onset of a project is essential to achieving buy-in and acquiring support. Public endorsement for the preferred alternative during the project initiation phase removes several barriers to delivering the project in construction and increases satisfaction with the final product.

By its very nature, ABC with short term road closures requires heightened public involvement throughout the project delivery process. For example, at the onset of the project, it's vital to ensure that affected communities are agreeable to a short term road closure to curtail public opposition during design and construction. In addition, engaging these communities early in the design phase helps to determine the optimum timing and duration for a short term road closure to mitigate associated traffic impacts to

community events, local businesses and emergency services. Once the project is in construction, project outreach is essential to keep roadway users and other key stakeholders informed of real time information prior to, during and following the short term closure so they can plan accordingly.

With an emphasis on transparency and implementing best practices for public involvement and outreach, many new strategies have been deployed by the ABP from project initiation throughout project outreach. For example, during the project initiation phase, the affected town and associated regional planning commission are asked to complete a "Community Questionnaire" to learn more about considerations for the timing and duration of a closure (annual public events, emergency services and potential adverse impacts to businesses), bike and pedestrian use, the best methods to reach out to the public and other design considerations. Scoping engineers use this information to determine if a closure is feasible and identify key stakeholders that should be engaged throughout the project along with the other important features referenced above. During the public "Alternatives Presentation", project managers from the ABP discuss how the community and regional considerations were used to develop the scope. In addition, to encourage active and meaningful public engagement during these initial public meetings, an audience response system is used to learn more about demographics of the meeting participants, acceptable closure durations, essential design considerations and overall satisfaction with the proposed alternative. Responses to the polling questions are recorded and uploaded to a SharePoint site that can be accessed by the public and other stakeholders at any time. This information helps to project manager to identify areas to focus on during the design phase and discuss at future meeting and is used to develop a public outreach plan.

Once the project moves into the design phase, ABP design project managers continue to coordinate with town officials, emergency services, local businesses and affected property owners on a regular basis to determine the optimum time and duration for a short term road closure along with the best way to accommodate emergency services. For most ABP projects, a project outreach coordinator is brought onboard during the final plan stage to implement project outreach strategies as the project moves into construction. Generally, project outreach includes a "Preclosure Public Information Meeting", weekly email updates two weeks prior to, during and following the closure period and the distribution of project factsheets (a two page document that includes project highlights including the purpose and need for the project, closure period and duration, regional detour map and contact information). For higher profile projects, a public viewing area is also established. Following the completion of the project, surveys are sent out to document satisfaction with: (1) public involvement in developing the alternative, (2) innovative construction methods, (3) traffic management and, (4) the quality of the information received prior to and during the closure.

Heightened public involvement and outreach has proven highly successful to the deployment of ABC and overall public satisfaction. On average, 80% of all respondents have been very satisfied with ABC even in communities that were initially opposed to this approach to project delivery. Overall, Vermont communities want to be involved in developing the scope of the project and work collaboratively with VTrans as opposed to feeling like something is being done to them.

DEVELOPING FUTURE LEADERS

In a time of funding uncertainty and continued deterioration of the existing transportation infrastructure along with other challenges developing future leaders that can adapt and think innovatively is critical to maintaining a reliable and efficient transportation network. ABC is changing the conventional mindset of delivering projects. Not only does ABC focus on innovation, efficiency and reducing or eliminating impacts to the landscape but also flips the paradigm to a customer oriented approach by minimizing traffic impacts and increasing the safety of the traveling public. This encourages transportation professionals to consider innovation as a standard course of action and optimize the use of available funding to maintain the network while adapting to changing trends.

THREE YEARS OF PROVEN PERFORMANCE

Since its inception in 2012, the ABP has delivered 28 projects totaling $71.3 million with another 17 projects under development. The program has gained significant momentum since the early stages of implementation and proven successful at expediting project delivery by reducing the standard design duration from 60 months for conventional projects down to 24 months. Due to these achievements, the program has also received the support of Vermont's legislative branch by enacting Act 153 which reduces the town share on a town highway project by 50 percent if the town chooses to close the road vs. installing a temporary bridge. The ABP has also been used to replace several legacy bridge projects programmed since the 1980's. Much of this success is attributed in large part to partnering with local communities.

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HARTFORD VERMONT BRIDGE REPLACEMENT SLIDE-IN BRIDGE CONSTRUCTION AND CM/GC PROJECT DELIVERY

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ABSTRACT

Narrow roadways with challenging landscapes make innovative bridge construction essential in many locations throughout Vermont. These challenges in conjunction with high traffic volumes drive decisions to use innovative solutions for both project delivery and construction. The Hartford I-91 Bridge replacement project is an example of innovative project delivery in conjunction with accelerated bridge construction as a means to successfully deliver and construct a complicated project with high public expectations for mobility.

INTRODUCTION

The Hartford project involves two complete bridge replacements (bridges 43N and 43S) on Vermont's eastern interstate I-91. These two Bridges carry significant traffic over US Route 5, also a high volume roadway, in the town of Hartford. Both bridges are three span structures with a pin and hanger type connections beyond the piers as shown in figure 1. These connections became problematic along the fascia beams and have required significant maintenance to address critical defects over the past 5 years. Because of the extensive maintenance repairs and a desire to remove bridges considered to be fracture critical from the Vermont Interstate system, the projects were programmed for replacement in 2012.

Figure 1: Fracture Critical Girder Splice Detail

PROJECT PLANNING

An appropriate project scope is essential to the success of any accelerated bridge project. Accelerated construction cannot be forced and road closures are not always an optimal solution. Often times the available detours are far too lengthy, ADT is too high or emergency services cannot tolerate a road closure. Early planning in the form of project constraints, resource identification and public outreach is a primary focus of the Structures Scoping Unit. This group works closely with all public and private stakeholders to develop appropriate project scopes for all Structures projects. The first step in project development is to determine the appropriate scope and the Hartford project was no exception.

ABC VS CONVENTIONAL CONSTRUCTION

While the substructures for both bridges were in good shape, a future project to widen US 5 and the desire for wider bridges on the Interstate system resulted in a project scope recommending full replacement of both bridges. However, many constraints exist as shown in figure 2 below, which steered the scoping effort away from conventional construction using traditional traffic control measures such as phasing or crossovers. The interchange for I-89/I-91 is located just south of the bridge which is located at an exit to a major US route. Furthermore, there are sharp curves, steep grades, and other unavoidable geographic constraints and a river crossing just north of the project. It was obvious we would need to get creative with the construction of these two bridges while focusing on maintaining mobility with minimal traffic

disruption. As the project progressed through scoping, ABC was determined to be the best solution for this site.

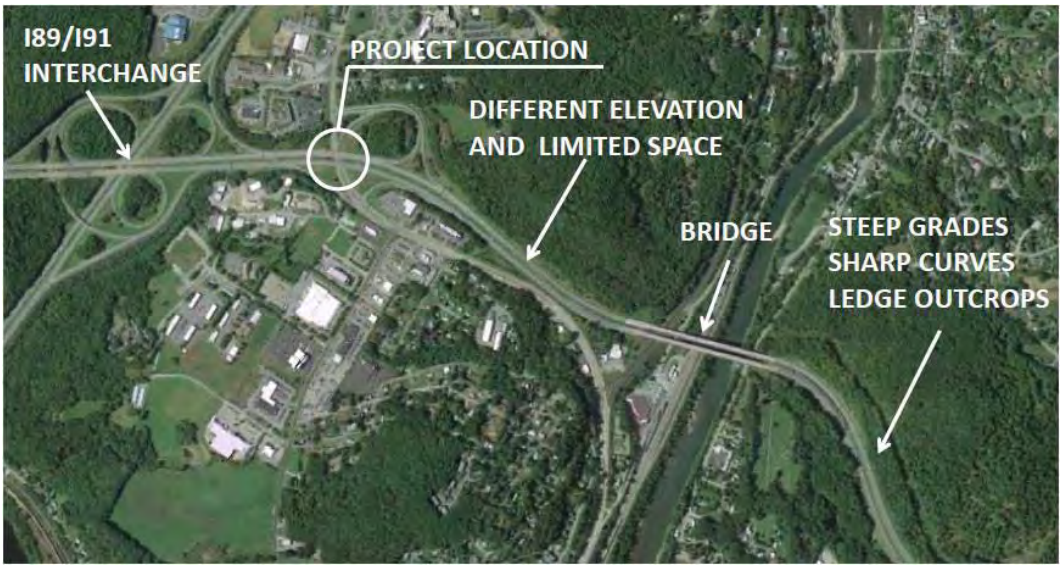

Figure 2: Project Location

VTrans referenced Table 1 from FHWA's Slide-In Bridge Construction Implementation guide to determine if slide-in bridge construction (SIBC) was an appropriate ABC technology for this project. There seemed to be enough geometric constraints prohibiting traditional maintenance of traffic but it is important to be able to construct new supports while maintaining traffic on the existing bridges. As it turned out, the geometry of the existing bridges was ideal for constructing new abutments below the bridges between the existing shoulder piers and existing abutments. This work could be done with minimal impacts to traffic mobility on both I-91 and US 5. While SIBC seemed feasible for the site other ABC superstructures were evaluated as well. Precast Bridge Units as well as Self Propelled Modular transporting the bridges were considered but discarded in favor of SIBC. Fortunately the state of Vermont has extensive right of way along the bridges which allowed for constructing new superstructures adjacent to the existing bridges. See figures 3 and 4. SIBC was the best bang for our buck.

Figure 3: Northbound bridge location

Figure 4: Southbound bridge location

MAINTENANCE OF TRAFFIC

Despite the construction method chosen, maintenance of traffic would still be a challenge during construction. VTrans was committed to minimizing impacts to road users and was determined to close each bridge for just one weekend to slide the new bridges into place. But this was a new concept and those being presented the plan were somewhat skeptical. Fortunately, there were many examples of where this technology had been used successfully and we were able to build confidence from success stories around the country. Based on commuting traffic counts it was determined we could close the interstate starting at 6:00 pm on a Friday night but had to open it by 6:00 am on the following Monday. Hartford is a high traffic area with many businesses, a hospital and a college in close proximity. Our primary goal was to maintain mobility for commuters on this route heading into Hartford.

As mentioned previously, US 5 is a major north-south route with traffic counts as high as those on the Interstate bridges that span it. Despite the ABC method for chosen for installing the new bridges, the traffic on US 5 would see much greater impacts than those using the interstate system above. There was no getting around the fact that there would be construction adjacent to the highway in four locations with the building of the new abutments as well as overhead work to build the new bridges in their temporary locations. Impacts to US 5 would exist for the entire summer and needed to be well planned and managed. It was apparent contractor input would be beneficial as we designed the project to best manage and minimize these impacts.

INNOVATIVE CONTRACTING

During scoping, project delivery method is always a consideration with recommendations made for alternative contracting when appropriate. Generally Vermont is a Design-Bid-Build state with most

projects being designed in house or by consultant and awarded to a pre-qualified low bidder. However, several years ago Vermont received legislative approval for and delivered some large scale projects using design build. This legislation was flexible as written and did not preclude the CM/GC contracting method from being used in Vermont. Because of our close relationship with FHWA and our commitment to the Everyday Counts Initiative (EDC), we had attended a workshop about Construction Manager/General Contractor (CM/GC) as an alternative contracting method and were interested in trying it. The Hartford project was a perfect candidate as it was utilizing innovative construction and the contractor's means and methods were crucial to succeeding and gaining public trust and acceptance of the project. With the support of the Vermont FHWA office we decided to pursue the CM/GC contracting model for the Hartford project. At the completion of the scoping process and the start of project design the stage was set for delivering the project as follows:

- The project would be designed by VTrans Accelerated Bridge Program in-house design team.
- VTrans would procure a CM/GC with heavy lift or slide experience with a commitment to ABC to work side by side with the design team during project development.
- Impacts to mobility under the bridges for both vehicles and pedestrians would be minimized and well managed.
- Each bridge on the interstate would be closed a maximum of 60 hours to replace the structures, with the closures occurring between 6:00 PM Friday through 6:00 AM Monday.
- VTrans would hire a professional project outreach coordinator to identify and work with all local and regional stakeholders.

CONSTRUCTION MANAGER / GENERAL CONTRACTOR (CM/GC)

To ensure a fiscally efficient and constructible project, VTrans opted to pursue the CM/GC contracting method to deliver and construct the Hartford project. Under the CM/GC model, the construction manager and their team provides input to the owners and designers regarding scheduling, pricing and construction phasing that helps the owner design a more constructible project. At approximately 90% design completion, the owner and the construction manager begin to negotiate a targeted maximum price for the construction of the project based on the project plans and schedule. If this price is acceptable to both parties, they execute a contract for construction services, and the construction manager becomes the general contractor. Procuring a contractor with notable bridge move experience for pre-construction and inevitably construction services significantly reduced the risk of constructability issues and concerns associated with traditional Design-Bid-Build delivery method. Furthermore, it built confidence in the design team given they had never designed such a complicated project.

CM/GC PROCUREMENT

At about 30% plans VTrans began the procurement of the CM/GC by issuing a Request for Proposal (RFP) outlining the goals of the project and procurement schedule. The selection was based on qualifications only as VTrans was concerned with finding and selecting a contractor with the appropriate ABC experience. Four firms sent in proposals outlining their approach to the project and how they would be able to meet the project goals as outlined in the RFP. The proposals were ranked by a committee and two firms were short listed for interviews. PCL Civil Constructors, Inc (PCL) was the winning firm with extensive experience ranging from barging bridges into place to sliding bridges. Furthermore, this firm had a successful track record with many ABC projects in high traffic areas and was bringing an experienced construction engineering team to work with VTrans engineers. PCL was contracted for preconstruction services for the Hartford project in January 2014.

DESIGNING FOR CONSTRUCTABILITY

Together, the Project Manager and designers from VTrans collaborated with the Project Manager and key personnel from PCL to design and prepare a solid construction contract. Partnering is essential for building trust in the team and delivering a constructible project. Every month a meeting was held to review project plans, specifications and consider constructability. Right from the start means and methods for construction were considered. A prime example of the benefits of partnering was experienced at the project kick off meeting. The foundation type was critical and while the 30% plans showed a Mechanical Stabilized earth wall with a shallow foundation abutment, PCL requested changing to a micro pile foundation. PCL was concerned about the amount of excavation needed to install wall reinforcement and the bracing that would be required at the existing abutments to accommodate these long strips. The design team agreed to the concept and began exploring micro piles as the foundation for both bridges. Because of the low headroom construction, it was essential to reach out to the subcontractor who would be installing them to determine an appropriate size of pile given the overhead constraint. This communication with subcontractors is another benefit of CM/GC as it allowed us to work with the company doing the work. Once this information was available design could be complete and the number of piles necessary was certain. PCL was able to show financial benefits of the change in foundation type and prove the construction schedule would be able to accommodate this method of construction.

Other design details and specifications were presented and discussed as a team throughout the remainder of the design phase. Subcontractors were brought in as necessary to ensure the project was heading down the right path both fiscally and constructability. Through plan reviews and meetings, appropriate modifications were made and the project plans progressed through final design.

MANAGING RISK THROUGH A RISK REGISTRY

All projects have risk associated with their construction. Identifying and managing these risks is a key component to the CM/GC process. As VTrans worked with PCL to design a constructible project, risks were identified that could negatively affect the schedule, compromise our project goals or add costs to construction. Generally a contractor builds the costs associated with such risk into the costs of a project. Any aspect of the construction seen as a risk will no doubt result in higher project costs. The CM/GC process allows both parties to identify and mitigate risks as a team. As risks are mitigated or removed, the cost to construct comes down. Figure 5 shows a risk registry used to identify and mitigate risk for the Hartford project. As the project progressed, risks were either removed or accepted with costs applied accordingly. Identifying risks allowed us to create contingency items that would remove or isolate the element of risk. For example, there were significant costs carried in the paving item for a Jet Dryer used to dry the pavement surface in the event there was rain during paving operations. This came to light during cost reviews and we were able to create a special item just for the Jet Dryer. As a result of the CM/GC process, this cost which was carried in the paving item was removed and the cost of paving came down.

ITEM	RISK TYPE	RISK ELEMENT	COMMENTS [Include Logic/Calculations of Cost Impact]
1	Design	Survey File - Inaccuracies or incomplete information in the survey file could lead to rework of the design	Verify that the survey file is accurate and complete. Survey centerline when in construction to verify roadway grades and limit of approach work
2	Construction	Underground Utilities - Plans provided by the Municipality did not show the underground utilities in the proper location. Dig safe during Boring operations located utilities and they are directly under the proposed abutment on BR 43N south abutment	Working on a relocation plan and permit for relocation. Plan for relocation to occur in Fall 2014 for cable fiberoptic. Waterline to have valves installed so water can be shut down at structure. Line will be relocated after construction. Existing pier footing to be partially removed to allow for line to be relocted.
3	Design	Traffic Congestion - There may be signal timing issues at the intersection of VT Rte 5 and Sykes Avenue. Traffic patterns will be altered through out the project with one temporary signal added at the southbound off ramp. During the northbound closure another temporary light will be added at the northbound off ramp	Requested signal timing study and will make adjustments to signal timing as required. Will also provide uniformed traffic officer to clear ques as required
4	Construction	Micropile installation - Drilling micro piles will be variable depending on bedrock locations. Additional borings have been obtained but there are still unknowns.	Include contingency value for difficult drilling conditions to use as required for Micropile item. This contingency will be included in Targeted Maximum Price.
5	Design	Route 5 improvements - The Municipality has future plans for developing the section of Route 5 under the bridges. While we are accomodating the future width of rte 5, we are not accomodating any other improvements	Add a permanent right hand turn lane for south bound on ramp. Remove south bound slip ramp.
6	Construction	During 30% design and pre-lim 60%, the NW footing was 4+ feet below existing footing.	PCL asked to drill down on what was forcing this design. Vtrans looked into and it was the large drainage swale elevation. Vtrans modified design by raising footing 3+ feet, raising the drainage profile and placing a curb to control the US5 runoff. This reduces the shoring required between the abutment and new footing and eliminates the shoring required between new footing and exiting pier.
7	Equipment	What if jacks or manifolds malfunction during bridge slide	PCL will investigate costs for complete backup set of jacks, manifolds, pumps, etc. Likely to have 2nd set on hand. Will dry run equipment days before closure as well
8	Construction	SOE walls - cost and schedule	PCL inquired about the stability of the existing abutment foundations and loose vermont soils. Vtrans confirmed 1.5:1 from bottom of abut footing is allowable. This will greatly reduce the amount of SOE walls and may eliminate completely on the south side of US5
9	Construction	Micropile - cost and schedule	Vtrans provided various options on micropile sizes and batters. PCL discussed with specialty subs to identify most cost effective recommendation
10	Construction	SOE walls - type	Micropile & lagging has greatest initial costs. Soil nailing & shotcrete has cheapest initial cost, but risks are substantially higher until system installed. During installation, large cuts exist that are unsupported and could result in settlement of I 91 under traffic. PCL will continue to investigate, but imagines VTrans would rather pay the costs than take the risk.

Figure 5: Risk Registry

INDEPENDENT CONSTRUCTION ESTIMATE (ICE)

An independent cost estimate (ICE) is performed to validate all cost estimates for the project. The purpose of an independent cost estimate is to ensure due diligence has been completed to determine a reasonable and fair cost to construct the project. In lieu of competitive bidding, this is the owners chance to be confident in what has been proposed regarding means and methods and the cost associated with the project. The ICE serves as a benchmark for evaluating how reasonable the contractor's proposed costs

are. As the project progresses through the design phase, evaluations and comparisons are made between the Engineer's estimate, the contractor's estimate and ICE.

VTrans procured Stanton Constructability Services, Inc to perform the ICE services for the Hartford project. Stanton was brought into the project later than the contractor which was not ideal. An important function for the ICE is to participate in constructability reviews and offer ideas for making the construction more efficient from both a scheduling and cost perspective. Ideally, the ICE is from the contracting community, understands production based estimating and can offer innovative construction solutions as the project is developed. The best case is the ICE adds value to construction activities resulting in the best means and method as well as best pricing. Bringing the ICE into the project at the same time as the CM/GC allows for the best partnering and sharing of ideas for construction. Both parties are vested in the project if they are both involved from the start.

VTrans scheduled three estimate submittals for evaluation and comparison. The first estimate was submitted at 60% plans, the second at 90% and the third at when the plans and provisions were complete. Meetings were held to determine the best format for reporting and to make sure indirect and overhead costs were not included in the pricing. While indirect costs such as engineering and construction administrative personnel and company overhead must be considered, it is important to be only looking at the cost to construct the project when comparing estimates. The indirect costs were reported in a separate category and were added to the items at the end of the process. The goal was for the ICE and CMGC to be within 5-10% of each other by contract time. If this was the case, VTrans would award the project to PCL to construct. If not, the project would be prepared for advertising.

The first estimate was as expected and was significantly different between all three parties with a variance of nearly 25% between the CMGC and the ICE. The engineer's estimate proved to be a challenge given the software used for estimating is based on average bid prices with average indirect costs and overhead included. Furthermore, the engineers doing the estimate just don't have the connections contractors have for obtaining quotes. The evaluation of the first estimate was blind, meaning only differences in items would be discussed. Neither party would know who was high or low just that there was a difference in pricing for the item. This worked pretty well and started discussions on means, methods, self-performed vs. subcontracted work. The second and third estimates were completed at 90% and 100% plans. These estimates were compared with books opened to discuss all aspects of pricing. Many hours were spent talking about production rates necessary to construct the project within the timeframe specified by the owner. Still there were significant differences in the estimates with a delta at the second submittal being 16%. By the time the final estimates were complete the delta was much closer at 6.5%. The comparison

of estimates was time consuming but was effective in drilling down into the means of constructing the project. On a couple of occasions, VTrans, PCL and the ICE actually scheduled a conference call with a subcontractor to determine best value in procurement.

TARGETED MAXIMUM PRICE:

In December 2014 VTrans and PCL agreed on a targeted maximum price for construction services. The final cost with indirect costs and overhead included was $11.1 million. Many unit prices were left in place with others becoming final plan quantities (FPQ) and would be paid per plan. Items like micro piles were left open to alleviate the risk to the contractor for extended depths to bedrock. While the project did not start out schedule driven, by the end we were very concerned with being able to order material such as structural steel and were not as confident with the final quantities to simply turn the project to lump sum.

PROJECT OUTREACH

Public outreach is essential to the success of any high profile project. Several public meetings were held with Town Officials, local residents, and local and regional stakeholders. PCL presented with VTrans as partners during design and continue to participate in public meetings as we move into construction. This solid partnership continued to provide assurance to all stakeholders on the innovative construction technique used and its positive effect on traffic concerns.

PROJECT CONSTRUCTION

Partnering Through Construction:

The understanding of the construction means and methods was maintained throughout the construction phase of the project. Ideas and preliminary drawings were shared during the preconstruction phase, leading to expedited reviews, fewer comments, and a team environment. The continuity of personnel from both VTrans and PCL helped transition from the design phase to the construction phase.

ABC Construction Methods:

The Hartford Lateral Slide project was an ABC project on multiple fronts. The pre-construction services contract was executed in February, 2014 with construction to be complete in October 2015, leaving 12 months for design and 8 months for construction. Removing and reconstructing two bridges during this time frame is difficult. The use of the CM/GC delivery method made this schedule possible. General construction planning occurred alongside the design and costing exercises during pre-construction, allowing for only the final details to be sorted out during the construction phase.

With schedule as a driving factor, innovative construction techniques were evaluated. Precast deck forms were selected for use on the project. Using the precast forms, time required for the installation and removal of deck forms was eliminated from the schedule. The third ABC method is the lateral slide method, which will be discussed further later.

ABC Foundations:

The foundations for the new structures were selected with constructability and schedule as the driving factors. In order to meet the schedule and the project requirements, the foundations were required to be installed underneath the existing structure between the abutment and the existing pier. The easiest and most cost effective way to install foundations was to excavate the slope to create access to the work area, and then install a micro pile foundation. The bedrock layer is near the surface making this a viable solution. The micro piles were then capped with a typical footing. The bedrock is near the surface at one of the 4 abutments, which allowed for a spread footing. A typical abutment wall with wing walls was constructed on top of the footing, and backfilled to approximately 6’ below the bottom of the existing steel beams. The abutment wall was cast within inches of the existing beams. Some portions of the wing were strategically left out to allow the new superstructure to slide through.

ABC Demolition:

All of the glory is in the slide. All of the risk is in the demolition. Demolition of the existing bridges during the closure weekend was identified early in the process as the riskiest operation because of the unknowns, and schedule impacts of the unknowns. To mitigate this risk, one lane of the existing bridge was allowed to be removed in advance of the closure weekend. This gave the team a chance to execute the demolition plan and troubleshoot any issues prior to shutting down I-91. This turned out to be a very smart mitigation technique, since the original demolition plan did not work.

I-91 closures were scheduled to occur prior to the seasonal traffic of the “leaf-peepers”. The light traffic during this time of the year allowed I-91N to remain in a lane closure for an additional week while a new demolition plan was developed. The demolition of the existing structures is tricky because of the suspended midspan. As previously mentioned, the bearings of this span are in poor condition, and is the driving reason for the replacement of the structure. Tipping of the backspan beams is the second concern. The third concern is with the composite action of the suspended span. Shear studs exist only in the suspended span, and the girders can barely support the concrete deck load without the composite action. And finally, the backspan deck was intermittently tied to the beams with form hangers. These concerns led to a detailed sequence of demolition.

1. Remove temporary barrier from bridge
2. Remove asphalt pavement/granite curb from bridge
3. Saw cut bridge deck, remove midspan deck panels
4. Remove approximately 40’ of bridge deck near piers with percussive demolition
5. Slab crab removal of backspan deck slabs
6. Removal of suspended span girders
7. Removal of open joint and finger joint
8. Removal of backspan girders
9. Removal of approach slab / abutment / piers

There are multiple constraints and subsequences requiring detailed engineering at each step. Safety is the primary concern, and with large sawblades, torches, lead paint, large equipment, fall hazards, etc., demolition during the closure was the most supervised activity. During each closure period, the bridge was successfully removed in approximately 18 hours.

ABC Lateral Slide:

As previously mentioned, the lateral slide is the glory of the operation. During the bridge closures, the lateral slide was one of the fastest operations. It took approximately 4-5 hours to slide the bridge 60 feet. The success of a slide is determined in the upfront planning and layout. There are two fundamental rules that have to be followed for layout: The tracks have to be parallel and have relative matching elevations along the track. Meeting this requirement is more difficult than it seems, as this has to accommodate mill tolerance, deflection, field tolerances, etc. Bridge decks are not very flexible and minor deviations in elevations could lead to cracking.

Selection of the slide system depends on the specific project. The two most common systems are rollers, and PTFE/Stainless slide systems. Each system has its advantages and disadvantages. A PTFE/stainless system was chosen for this project due to its ability to be adjusted easily in any plan dimension. The hydraulics were set up in such a way that the bridge could be steered as we moved it by simply starting one side ahead of the other. The disadvantage to this slide system is that it is very sensitive to the cleanliness and smoothness of the track. It’s important that all dirt, debris, weld splatter, grease, oil, etc. be removed both from the PTFE pads and the stainless prior to the slide.

A common concern for lateral slide systems is the lateral forces that need to be restrained from pushing or pulling the bridge. This project used a system that maintained all of the forces internal to the system, resulting in internal axial loads instead of external horizontal loads. Accounting for this axial load in the

support beam did not require any increase in section, reducing the cost in comparison to an externally braced system. The system consisted of a W section with a continuous stainless surface welded to the top. A slide beam with PTFE pads located at girder locations slides along the top of the support beam. The 18” hydraulic jack pushes the slide beam while being restrained by tabs on the support beam. This system also keeps all lateral forces out of the new superstructure. Impact factors are negligible since the superstructure moves at such a slow rate.

ABC Roadway:

If all of the risk is in the demolition, and all of the glory is in the slide, then all of the schedule is in the roadway construction. There is one operation that is working continuously throughout the closure period, and that is the roadway. As much as possible, backfill was installed prior to the closure period to reduce the workload during the weekend. One method used to expedite the roadway construction was the use of precast approach slabs. The approach slab was constructed in two pieces at each end of the bridge to allow for crane erection. The slabs had block outs placed in them that slid over dowel bars in the back wall. The closure between the slabs and the voids in the block outs were filled during the closure period with a rapid set concrete mix that obtained strength in less than two hours. After the fill and approach slabs were set, the roadway was paved, striped, and opened to traffic.

Closure Cooperation:

It cannot be stated enough how important it is that everyone is working toward the same goal during an operation such as this one. It was apparent that all parties, VTrans, EIV, PCL and all of the subcontractors were committed to opening the bridge before Monday morning at 6:00 A.M. With this cooperation, everyone was invested in the outcome and worked together to insure this goal was met. Both bridges we opened to traffic ahead of schedule.

LESSONS LEARNED

Design

1. Procure the ICE early and bring them into the team at the same time as the CM/GC. Partnering is the key to success and the ICE should feel as vested in the project as the CM/GC. Since the ICE is from the contracting community, they can have significant input into the means and methods for construction the project
2. Estimating is difficult on a project involving a new technology and was further complicated by the CM/GC process. Engineers generally do estimates based on bid history and are not equip to perform production based estimates.

3. Public outreach is essential. Using professionals to perform outreach throughout the design construction phase assures high quality and constant communications. Stakeholders were writing in to say how much they appreciated the information.
4. Both the CM/GC and ICE were not local to Vermont and had to travel extensively for meetings. More onsite meetings with the entire team would have been better for overall constructability. While there were many meetings held on line with project plans, there were only a couple that included all key personnel including the project superintendent.
5. The Superintendent was changed out during the design phase. While the new Superintendent was deemed acceptable, he never was brought to the site and involved in means and methods. Some were made during construction as a result.

Construction

1. Heavy traffic is in the eye of the beholder, and has a different meaning in different areas. Know what this means in the area you are working.
2. Make sure the effects of dimensional tolerances are understood. Additional tolerance is not always the answer.
3. Shut the underneath road down during demolition. It is faster and less disruption than intermittent openings.
4. Beware of items such as formwork accessories that are not in the existing plans.
5. Find a way to mockup or test means and methods prior to a closure.

ACCELERATED BRIDGE CONSTRUCTION STUDIES FOR SIX CITY OF ASTORIA BRIDGES

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The City of Astoria is located in northwest of the State of Oregon, has a very rich history and is one of the most popular tourism attraction areas of the State of Oregon. One of the City's attraction area is Astoria Riverwalk with many restaurants, bars and business. There is a popular trolley service starts in month of May. Therefore, there is a great desire and demand to have no impact to trolley services and businesses during tourism season.

The City was evaluating replacement of six structurally deficient bridges which are posted for 3 tons load limit and are located in Columbia basin which may have very low water clearance during tidal time. These structurally deficient bridges are supported by decayed timber piles (Figure 1). The load limits severely limited functionality of bridges and ease of future development. These bridges are supporting pedestrians, vehicular traffic and trolley but trolley is supported on an independent timber pile trestle structure within city's right of way (Figure 2). The bridges are connecting the end of 6th through 11th streets to piers on the Columbia River. Design work is anticipated to start in 2015 and two season construction scheduled for years 2016 and 2017.

Figure 1, Right showing existing timber piles supported bridge and left is showing utilities supported on eleven street bridge.

Replacement of these bridges is limited to three bridges at a time for example, 6th, 8th and 10th streets or 7th, 9th and 11th streets to provide access to the surrounding businesses. There are other challenges such as

utilities, environmental concern, wetland and in water work window. Only one of the bridges does not have existing utility but all other bridges supporting utilities for example 11th street bridge has eleven utility lines (Figure 1). All surrounding businesses are dependent to these utility and need to have them functional all the time. Design and construction complexity combined with in water work window, trolley service and not impacting businesses during tourism season was promoting to reducing two season constriction time which was originally estimated to one season construction in minimizing impact to business during tourism season.

Figure 1, Left is bridges site map and right picture is showing bridge location and trolley rails and a commercial building adjacent to the bridge.

It was assumed the replaced bridges would be one of the modern pile supported superstructures. However driving piles close to existing old structures imposed risk and liabilities. Therefore there was a need for further investigation on selection of substructure type which does not have impact to old buildings and does not require longer construction time. Also superstructure construction type to minimize construction time for replacement of all bridges within one season construction time deemed necessary and shallow superstructure because of low water clearance. Another question was; if there is a need to have two independents bridges in which one just supporting trolley rails to stay consistent with existing construction (Figure 3).

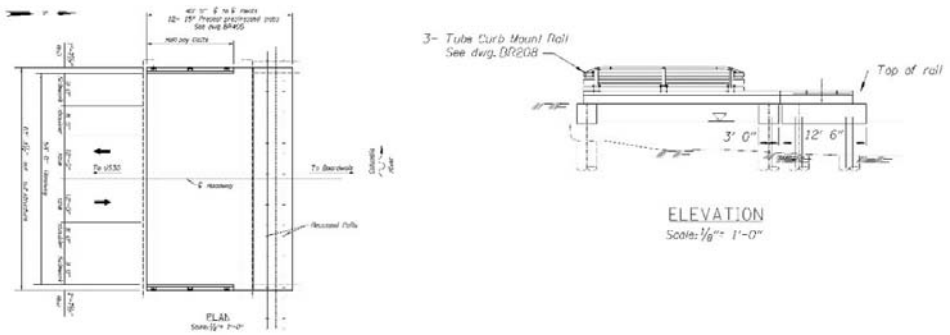

Figure 3. Plan and elevation of two independent bridges in which one is serving trolley.

It was evident the traditional construction does not satisfy all requirements of the project. Therefore, it was evident the accelerated construction is the solution for such complex project. This presentation will cover all types of construction that was studied and is not limited to the accelerated bridge construction.

WEEKEND REPLACEMENT OF SR 0581 BRIDGE SUPERSTRUCTURES

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ABSRACT

The purpose of this project was to replace the PA 581 twin bridge superstructures over 10th Street near the City of Harrisburg, Pennsylvania; promote flexibility using a design-build contract with an accelerated bridge construction concept to minimize traffic disruptions; promote traffic and workers safety; and improve quality and durability of the new bridge superstructures. The bridge superstructures were replaced using only four weekend traffic detours (two weekend detours for each direction). This paper provides a brief overview of the maintenance and protection of traffic, public outreach program, construction of the twin bridge superstructures using contract specified materials, and lessons learned throughout the design/build process.

INTRODUCTION

The existing twin, three-span continuous rolled steel beam bridges that carry PA 581 traffic over 10th Street in a suburb of Harrisburg, Pa., were originally built in 1960. The existing bridges did not provide sufficient vertical clearance to 10th Street and the superstructures were significantly deteriorated, classified as structurally deficient and requiring replacement. The clear deck width of the westbound and eastbound superstructures are 32'-9" and 46'-9", respectively and overall bridge length is 121' (Fig. 1 and 2). The eastbound deck is wider than the westbound deck because of an extra lane for the off-ramp from EB PA 581 to southbound Interstate 83.

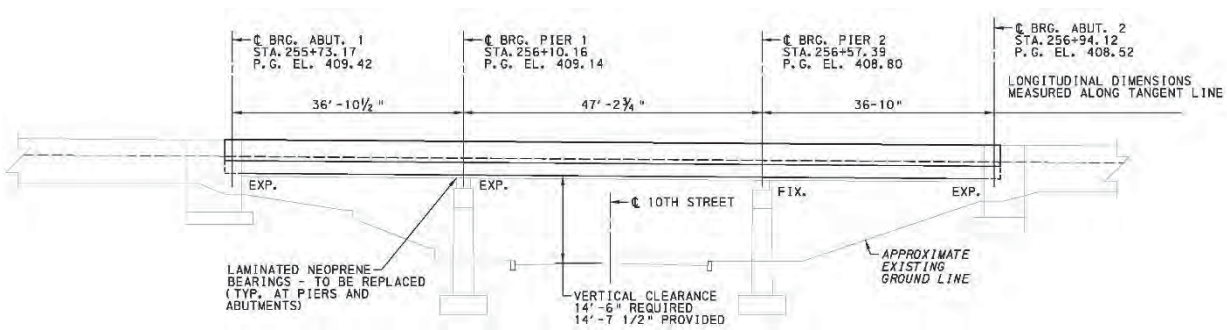

Figure 1. Typical Bridge Cross Section

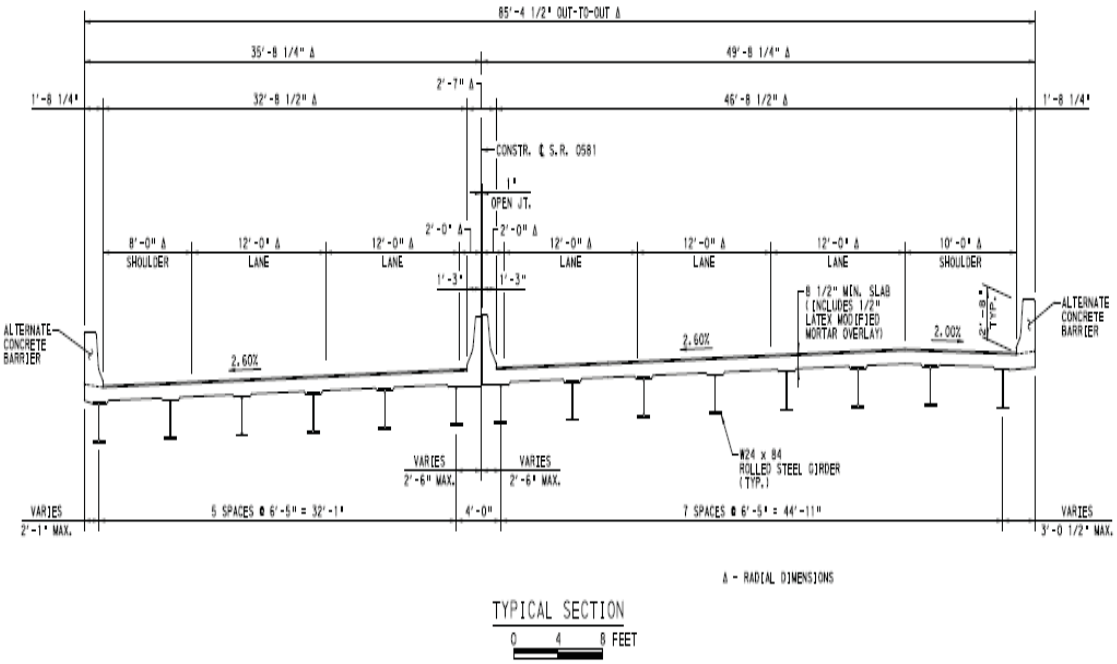

Figure 2. Typical Bridge Cross Section

The two options evaluated during the initial project development phase were replacing the superstructure using normal staged construction or using accelerated bridge construction techniques. The staged construction option would have required half-width construction with single lane traffic patterns on westbound PA 581 and three-phase construction with two lanes of traffic for eastbound PA 581 Eastbound. The schedule for phased construction would have required a minimum of two construction seasons to complete, resulting in significant traffic congestion and safety concerns, especially during the morning and evening rush hour commutes. To minimize these concerns, the Pennsylvania Department of Transportation (PennDOT) decided the best option for this particular location was to proceed using accelerated bridge construction techniques.

DESIGN

This project was bid in 2013 with a design/build component to allow innovation and flexibility for contractors, while maintaining full responsibility to the design/build team. The major components of low bid cost breakdown were as follows:

Design Items -

Bridge Design- $210,000

Maintenance and Protection of Traffic (MPT) Design - $50,000

Construction Items -

Bridge - $2,600,000

MPT - $125,000

Based on actual costs from previous superstructure replacement projects, PennDOT estimated the cost for the superstructure replacements using conventional means was in the range of $150-$200/SF of the bridge deck area. Replacement cost for these superstructures would have been in the range of $1.5 million to $2 million, but construction would have taken almost two years and significant traffic congestion during peak hours would likely have occurred. Also, since safety of the traveling public is a primary consideration during the project development process, concerns were raised with the extended construction time and substantial traffic volumes (86,000 vehicles per day). The estimated premium cost of $600,000 for accelerated construction was well justified based on safety concerns and the unpredictable cost of a serious incident during construction at this location.

The bid package included criteria to be used for design and construction of the new bridge superstructures. Some of the key criteria included:

- Minimum of two girders prefabricated unit, precast integral deck and girders.
- No non-integral precast bridge barriers for outside fascia barriers.
- Minimum 28-day concrete compressive strength of 5,000 psi for precast deck concrete.
- PennDOT approved cementitious, rapid setting, structural repair concrete not containing magnesium phosphate with a minimum 28-day compressive strength of 4,000 psi for deck closure or pocket pours.
- Retention of the existing strip seal retainer in the existing backwalls. New strip seal gland at each expansion joints.
- Use of high early strength latex modified concrete was specified to seal surface of the precast deck elements and achieve a smooth riding surface.
- New superstructure dead loads must be within 10% of the existing superstructure dead loads, and no change in bearing fixity was permitted.

The design developed by the design/build team proposed pre-casting new superstructure segments off-site. A 3D model of the existing structures were developed using a stationary LIDAR unit as well as conventional surveying techniques to ensure a proper fit between the portions of the existing bridges that remained with the new superstructures. Each precast segment would consist of two steel beams and concrete deck, and would span the entire 120’ – 11 ¼” between abutments. The depth of the steel beams used would be 6-inches shorter to increase the overhead clearance over 10th Street from 13’-11” to 14’-7”.

These sections would be shipped to the site and erected into place. Accelerated concrete would be placed to tie the segments together, and a latex overlay would be placed over the entire surface.

MAINTENANCE AND PROTECTION OF TRAFFIC DURING CONSTRUCTION

Maintenance and Protection of Traffic during construction consisted of maintaining all lanes of normal unrestricted traffic for the life of the project on PA 581 between 6 AM and 9 PM, except for two weekend closures for westbound PA 581 traffic and two weekend closures for eastbound PA 581 traffic for the superstructure replacement activities. These weekend detours were permitted between 9 PM on Friday and 6 AM on Monday on non-holiday weekends. Only one direction of PA 581 traffic could be detoured at a time. A detour for 10th Street traffic was also permitted during the weekend closures on PA 581.

Road user liquidated damages were specified for each hour or part thereof if lanes were not opened to unrestricted traffic flow outside the allowed time periods (Table 1).

Table 1. Route, Detour Length, and Road Users' Damages/Hour

Route	Detour Length (mile)	Road Users' Damages/Hour
PA 581 Eastbound	17.75	$14,500
PA 581 Westbound	17.75 and 4.40	$19,000
10th Street	1.50	$ 220

Road Users' damages for westbound PA 581 were higher than eastbound PA 581 because there were two separate detour routes, one of which utilized a local street network (fig. 3).

Figure 3. Capital Beltway

A public meeting and separate meetings with police and emergency management officials were held prior to the start of construction activities as per the contract requirements. In addition, assistance of the Pennsylvania State Police to provide queue protection was mandatory during the detour and lane closure periods and PennDOT directly reimbursed the State Police for costs associated for their services. PennDOT and the contractor prepared and distributed flyers describing in nontechnical terms the anticipated work sequence and required traffic control to adjacent residents and businesses.

PennDOTs regional traffic management center (TMC) was fully engaged during implementation of the detours, updating changeable message signs within and outside project limits upon timely notification by the contractor and PennDOT construction personnel.

CONSTRUCTION SEQUENCE AND ERECTION PLAN

A detailed construction sequence was developed, outlining the methods and schedule of operations to replace the superstructures in an allowable 57-hour window. Several meetings between PennDOT's engineers and the contractor's engineers were held to review these plans and identify potential problem areas. One of the major challenges was lifting the very large precast sections and setting them on the existing substructure. Each section was over 121' long and had to be lifted from eight separate lifting points to ensure that no portion of the new superstructure was overstressed during erection.

Total lifting weight, centroid, and weight distribution per beam for each section of removal of the existing superstructure and new superstructure were calculated (fig. 4). Since the eastbound assembly No. 4 weighed the most (278,182 pounds with rigging) and was not symmetrical about its longitudinal axis, it was used for the rigging component design and selection of the crane (Table 2 and 3. The selected crane was a Liebherr LR 1300 SX crawler with 144' boom length. Lifting capacity of the hook was 150 tons. Also, chain hoists with load cells were used to adjust the load so that loads were evenly distributed and enable the assembly to remain plumb during erection.

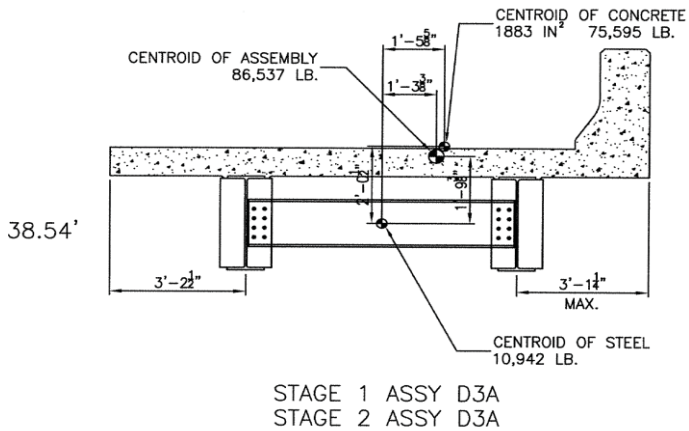

Figure 4. Centroid of Stage Assembly

Table 2. Summary of Lifting Weight for Removal

Designated Piece	Steel Weight (lbs.)	Concrete Weight (lbs.)	Total Weight (lbs.)	Total Lift Weight (lbs.)
D1A	10,942	70,818	81,670	91,836
D1M	11,201	83,111	94,312	104,388
D2A	10,942	55,803	66,745	76,821
D2M	11,201	65,490	76,691	86,767
D3A	10,942	75,595	86,537	96,613
D3M	11,201	88,717	99,918	109,994

Table 3. Summary of Lifting Weight for New superstructure

Stage	Assembly	Assembly Weight (lbs.)	Rigging Weight (lbs.)	Total Lift Weight (lbs.)
Eastbound	1	224,130	14,602	238,732
Eastbound	2	190,197	14,602	204,799
Eastbound	3	190,626	14,602	205,228
Eastbound	4	263,580	14,602	278,182
Westbound	1	227,803	14,602	242,405
Westbound	2	199,642	14,602	214,244
Westbound	3	249,650	14,602	264,252

CONSTRUCTION

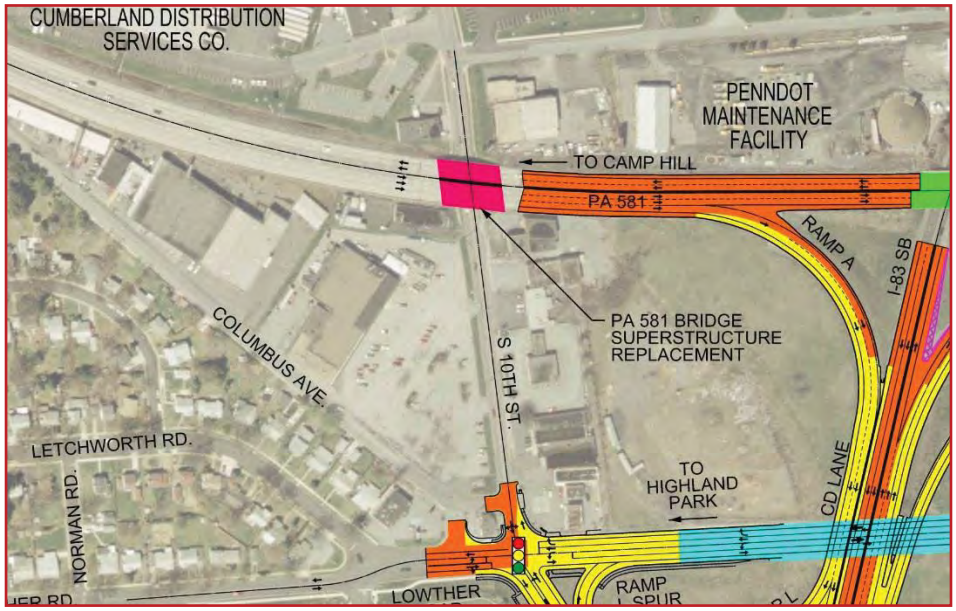

Figure 5.The Existing Bridge Location, Precasting Location, and Transportation Route

The new superstructure segments were pre-casted near the existing bridge in the southwest quadrant of the Interstate 83 / PA 581 Interchange (fig. 5). Each segment would consist of two steel beams and concrete deck and would span the entire 120’ – 11 ¼” between abutments. The segments were cast in a conventional manner on top of temporary substructure units consisted of compacted aggregate, timber cribbing, precast barriers, and steel bolsters (fig. 6 and 7). Longitudinal and transverse closure pour joints were blocked out during deck concrete placement. After deck concrete placement, the steel diaphragms between the segments were removed and the stay-in-place deck forms cut to separate the individual segments. Steel lifting lugs were welded to the new steel beams at the abutments, and four 1” diameter 150 ksi all thread bars were used for the installation of lifting lugs at the pier beam lifting locations. Since the pier lifting lugs could not be installed at the pier bearing locations because they could conflict with the bearings during erection, the pier lugs were shifted by 3’ inboard and stiffeners were installed at these locations. Also, lugs were slightly skewed to match the angle of the slings. Due to the unsymmetrical shape of the segments, chain hoists and load cells were used to monitor and adjust load distribution to ensure that the loads were properly balanced and the segment remained plumb during erection.

The three-span, continuous, two-beam units were placed on transport trailers in advance of the actual moving date. Self -Propelled Modular Transporters (SPMT) were not utilized on this project due to space constrains along the delivery corridor. Once the traffic detour was in place, the existing superstructure was removed in sections by longitudinal and transverse saw cutting of concrete deck and flame cutting the

steel beams. Placement of the first precast unit went as per plan. However, the other units required some minimal bending of the transverse deck reinforcement bars in a slightly upward direction in order to pass the reinforcement from the adjacent section. These bars were bent back to their original position after placement of the precast segments, and the bars were tied providing a minimum lap of 2’-1”. The formwork was then installed in the longitudinal closure pours and the transverse joint at each abutment. Accelerated concrete was then placed in the closure pour areas, consolidated, and finished. Once this concrete attained the required compressive strength of 2,600 psi, the bridge deck was opened to traffic.

On a subsequent weekend, traffic on PA 581 was again detoured for placement of a latex modified concrete overlay. The new superstructure and closure pours were scarified, and accelerated latex modified concrete placed at a minimum depth of 1 ¼”. Due to the rapid setting of the accelerated latex material, each deck was overlaid one half at a time to ensure that the overlay material could be placed and finished in a plastic state. Once the latex modified concrete achieved a compressive strength of 3,000 psi, the bridge was reopened to traffic.

Figure 6. Temporary Supports for Precasting of New Superstructure

Figure 7. Partially Constructed Superstructure

LESSONS LEARNED

Reuse of the Existing Bearings:

Since the existing bearings were in good condition, the contract permitted reuse of these bearings. During the superstructure placement activities, reuse of the existing bearings created some vertical and horizontal alignment difficulties. To close minor gaps between bottom of the sole plate and bearing assembly, the use of coated shim plates was required. Also, horizontal alignment variations required acceptance of minor deviations in optimal bearing edge distances. For future projects, we will consider including complete replacement of the existing bearings.

Joint Width and Joint Closure Material:

When this project was bid, the steel fibers required in Ultra High Performance Concrete (UHPC) were not available in the United State. As a result, UHPC was not specified for use on this project. The use of accelerated cement concrete in lieu of UHPC for the closure pours required a full lap length on the reinforcement bars, which resulted in a wider closure pour dimension and contributed to interference of reinforcing bars during erection of the precast units. To limit the closure pour width, the use of UHPC is highly desirable. However, the required curing time to obtain necessary compressive strength for UHPC is longer than accelerated cement concrete and should be accounted for in the accelerated construction schedule.

Expansion Joint:

The existing strip steel dam extrusion was relatively new on this bridge. In order to limit the reconstruction time, retention of the half of the steel extrusion in the backwall was specified in the

contract. The retention of the steel extrusion required cleaning out the old neoprene gland material from the extrusion, and setting the new half of the extrusion required significant time and effort to match existing grade. On future projects of this nature, we expect to require removal of the top portion of the backwall and complete replacement of the strip seal dam extrusion.

Rapid Set Latex Modified Concrete-Mobile Mix:

The contractor utilized an accelerated latex modified concrete mix with a citric acid retarding admixture that achieved the required compressive strength of 3,000 psi in approximately four hours. The use of the citric acid retarding admixture extended the workable time of the mix, allowing the contractor time to place and finish the material. The initial load that was batched on-site did not remain workable for a sufficient amount of time and required removal of the placed latex material from the bridge deck. Subsequently, the amount of citric acid admixture was adjusted to extend the workability of the material. Due to the significant effect that air temperature has on the workability of this type of mix, it may be beneficial to conduct trial placements at various ambient air temperatures to determine the amount of citric acid (or other retarding admixture) to utilize.

Anticipate Problems and Develop Contingencies:

The tight schedule constraints and significant road users liquidated damages (RULDs) typically associated with this type of construction require significant planning and scheduling. Due to the unpredictable nature of construction in general, part of this planning should include what to do if things go wrong. Do you have a contingency plan in case of an equipment breakdown? What if there is a fit issue with one of the precast segments? We suggest discussing as many scenarios as possible before beginning construction, and developing action plans before something goes wrong.

ACKNOWLEDGEMENTS

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A special thanks to Brian Moore, P.E., Construction Manager, for this project and Gregory Penny, the PennDOT District 8-0 Community Relations Coordinator for editing this paper.

UPDATE ON PENNDOT'S PUBLIC-PRIVATE PARTNERSHIP (P3) RAPID BRIDGE REPLACEMENT (RBR) PROJECT

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BACKGROUND

PennDOT's Public-Private Partnership (P3) Rapid Bridge Replacement (RBR) project is designed to bolster PennDOT's ongoing effort to address the state's nearly 4,000 Structurally Deficient (SD) bridges. With the P3 approach, PennDOT will replace 558 SD bridges around the state more quickly; achieve efficiencies for taxpayers; and minimize the impact on the traveling public. The initiative was approved by the Public-Private Transportation Partnership Board in September 2013. In October 2014, Plenary Walsh Keystone Partners was selected as PennDOT's private partner. Plenary Walsh and its team of at least 11 Pennsylvania-based subcontractors will manage the bridges' design, construction and maintenance for 25 years after construction is complete under the P3 contract. Construction began in the summer of 2015 and all 558 bridges are planned to be replaced within 36 months.

UPDATE

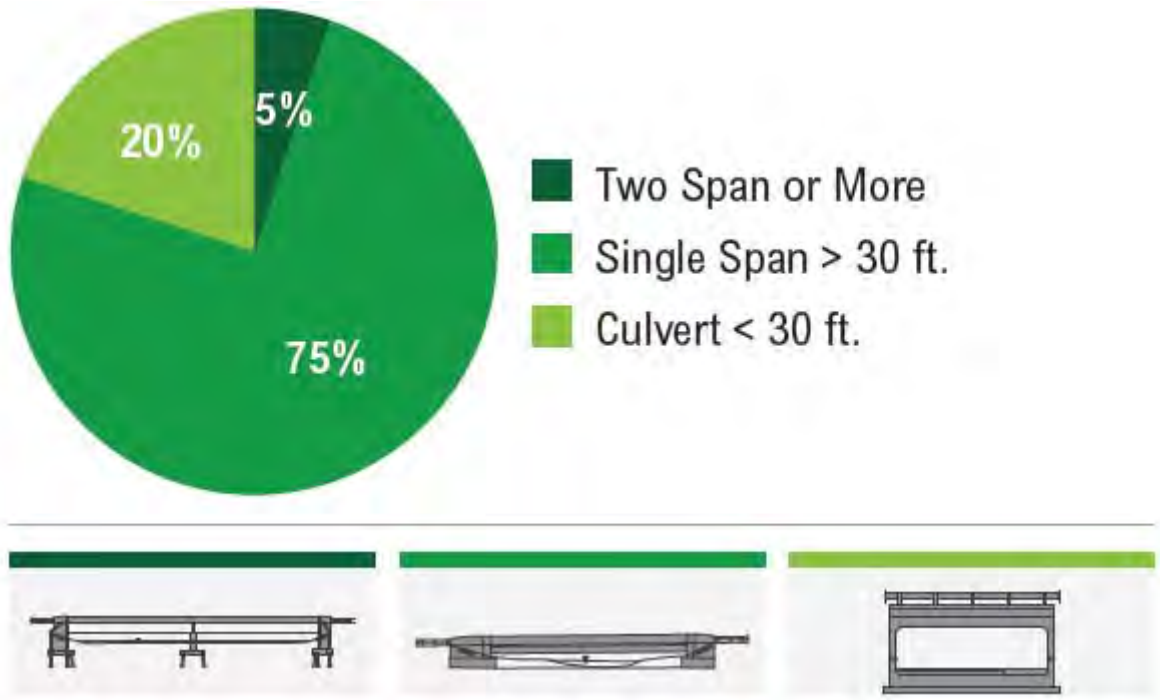
Design submission reviews have been underway since February 2015. Construction on the first bridges started in June 2015. As of September 10, 2015, NTP3 (commencement of construction) has been given on 62 Early Completion Bridges (ECBs). ECBs are similar to Design/Build, PennDOT provided TS&L, H&H, NEPA, ROW, Utility Clearance, and Permits; Development Entity performs final design. ECBs were intended to be constructed in 2015. Construction has started on 47 structures, and has been completed on 3 structures so far. For the Remaining Eligible Bridges (REBs) PennDOT provided scoping documents, minimum Bridge width, detour or staged, and 2 borings per bridge; Development entity to perform NEPA, TS&L, H&H, survey, ROW Plan, Permits, and Final Design. Over 200 bridges will be constructed in both 2016 and 2017.

There are 115 bridges currently scheduled with traffic restrictions < 35 days. A few of them that are scheduled to be constructed this year are precast box culverts. Anticipated Accelerated Bridge Construction to be used during the P3 RBR project includes Precast substructures, Precast deck panels, combined precast/prefab deck and girders, precast culverts, Integral abutments, Bridge bundling, and streamlining of the process.

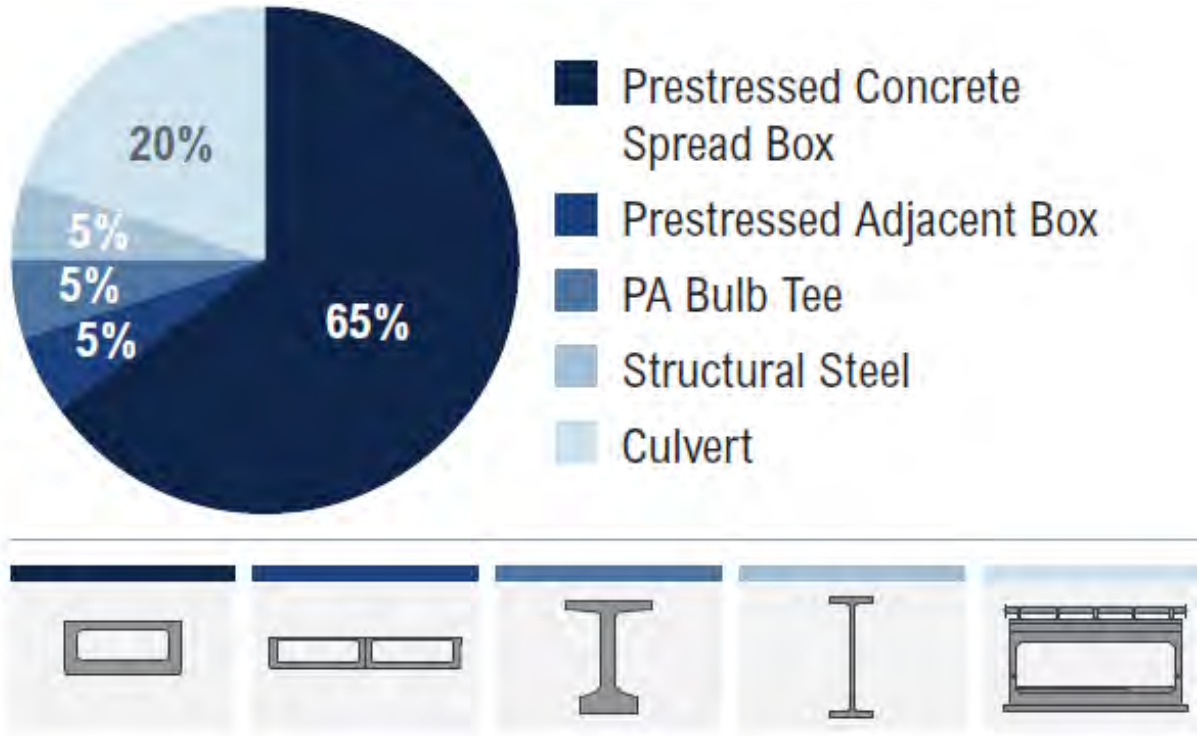
Ten Bridge standard designs will be used: Abutment Standard, Vertical Barrier Standard, Bridge Approach Slab Standard, Bridge Bearing Standard, Precast Box Culvert Standard, Foundation Design Parameters and Model Assumptions Standards, Roadway Special Details Standard, Spread Box Beam Standard, and Flume Detail Standard.

Bridge types for the 558 bridges are broken down as 398 P/S spread box beam bridges, 120 box culverts, 20 steel plate girders, 14 P/S bulb tee, and 6 adjacent box beam bridges. The following graphics show approximate breakdowns of data for these bridges:

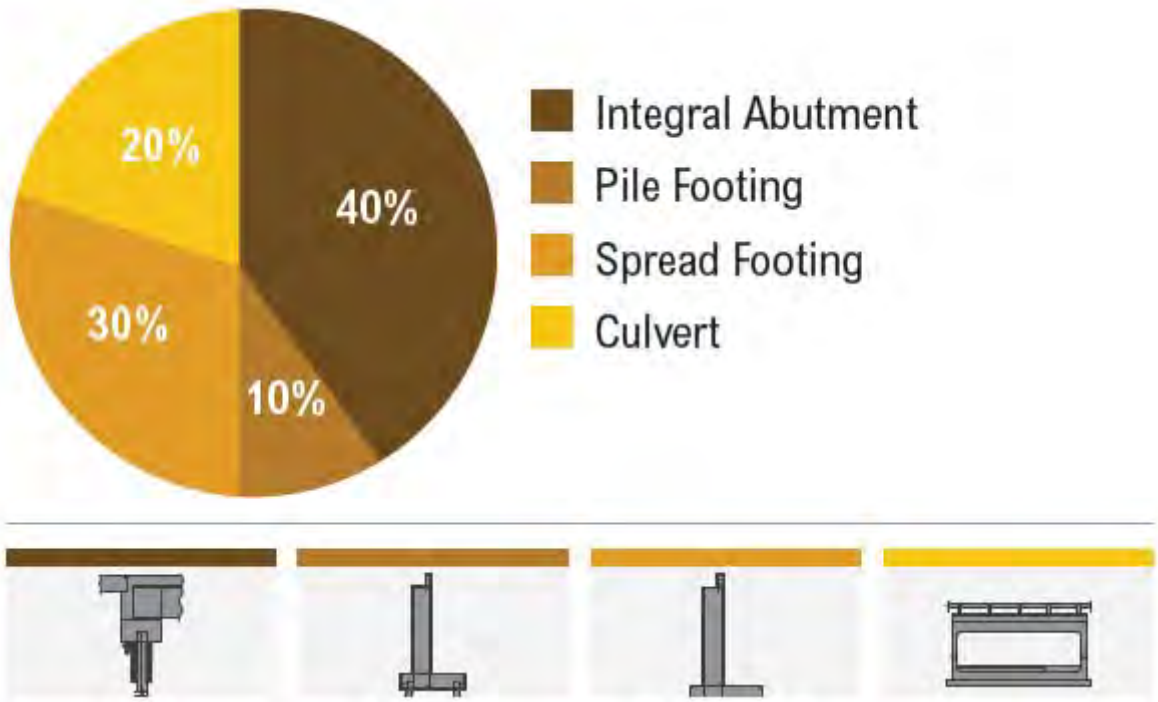
Replacement Structure Profile



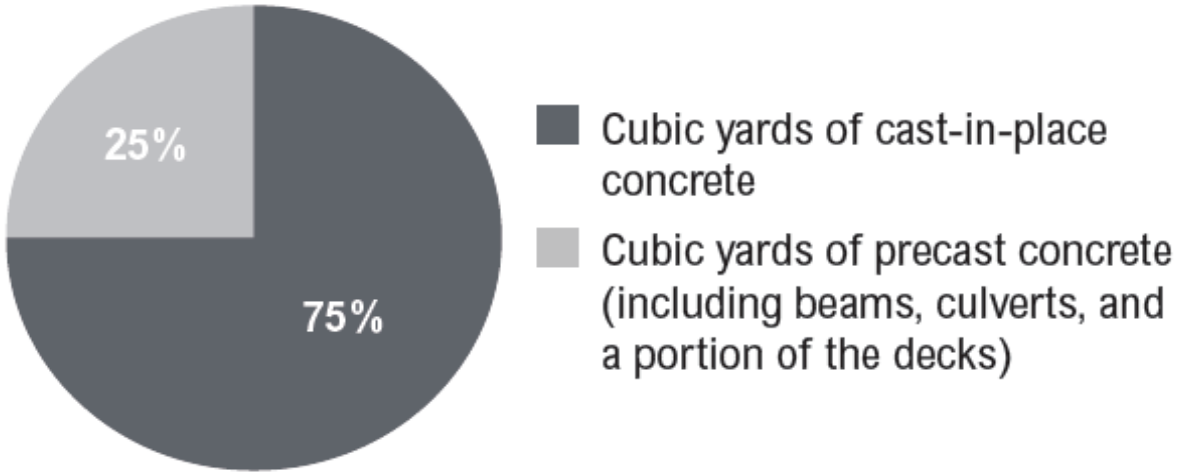
Proposed Superstructure Type



SUBSTRUCTURE TYPES



BRIDGE CONCRETE ELEMENTS



ADAPTATION AND EVOLUTION OF DESIGN DETAILS FOR IOWA DOT MODULAR BRIDGE CONSTRUCTION

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ABSTRACT

Introduction

To assess the viability of various accelerated construction strategies, and to cultivate the experience necessary to support sound policy decisions, the Iowa Department of Transportation (DOT) has engaged in a series of accelerated bridge construction (ABC) demonstration projects. After successfully completing several projects utilizing an assortment of ABC methods (1), in 2011 the Department demonstrated and showcased the use of modular prefabricated bridge elements and systems (PBES) for ABC. Iowa DOT's subsequent demonstration of the modular PBES technology, scheduled for completion in 2015, will incorporate second-generation details that have been designed to satisfy project-specific demands and incorporate lessons learned from the first modular PBES demonstration project.

IOWA DOT MODULAR CONSTRUCTION EXPERIENCE

US 6 over Keg Creek

In 2011, Iowa DOT hosted a national ABC demonstration project as a part of the Transportation Research Board (TRB) Strategic Highway Research Program 2 (SHRP 2) (2). The demonstration project, located on US 6 over Keg Creek in Pottawattamie County (3), was designed and developed by the SHRP 2 R04 project team. The three span, 0° skew, 204'-6 x 44'-0 bridge project featured a modular decked beam superstructure joined with ultra-high performance concrete (UHPC), and a substructure system extensively incorporating precast concrete elements. The ABC bridge replacement operations were completed under a two week road closure period in late 2011.

IA 92 over Little Silver Creek

Iowa DOT's current objective in its evaluation of the modular PBES method is to investigate how this technology might be adapted and standardized for more mainstream application on the State Highway System. In participation with the Federal Highway Administration (FHWA) Accelerated Innovation Deployment (AID) program, Iowa DOT identified a bridge replacement project that would both benefit from, and add benefit to, the Department's experience with modular PBES construction. Located on IA

92 over Little Silver Creek in Pottawattamie County (4), the three span, 20° skew, 234’-0 x 44’-0 bridge replacement project is scheduled for completion under a three week road closure in late 2015.

IOWA DOT MODULAR CONSTRUCTION DETAILS

Decked Beam Modules

The Keg Creek and Little Silver Creek projects both utilized similar details for their modular decked beam superstructures. With comparable span arrangements and identical roadway widths, both structures were suited to a configuration of 18 single-span decked modules per bridge, arranged 6 wide per span by 3 spans long. Each module is comprised of a longitudinal section of concrete bridge deck, approximately 7’-0 in width, cast on a pair of rolled steel beams. For the Little Silver Creek project, the section size of the rolled steel beams was increased to accommodate longer span lengths, and the modules were cambered and skewed to accommodate the more complex site geometry.

Superstructure Longitudinal Joints

Both projects utilized UHPC for the longitudinal connections between superstructure modules, favored for its superior bond and ability to develop reinforcing steel splices over short distances (5). The keyed longitudinal joints contain the transverse reinforcing steel splices required to develop continuity between adjacent modules, interlaced with longitudinal distribution reinforcement.

Relying heavily on the prevailing design procedures for conventional concrete construction, the longitudinal UHPC joints prototyped on the Keg Creek project were detailed with relatively congested reinforcing steel and narrow joint openings. Leveraging more contemporary research findings (6), the joints for the Little Silver Creek project were designed with simpler reinforcing splices, reduced reinforcing steel congestion, and eased construction tolerances.

Superstructure Transverse Joints and Continuity Details

The Keg Creek and Little Silver Creek projects were both designed to accommodate bending stresses resulting from live load continuity over the piers.

The early design strategy for the Keg Creek project was to rely upon narrow UHPC joints to transversely connect the superstructure modules and resist the tensile components of live load continuity. Laboratory validation efforts raised some uncertainty regarding the serviceability of this design (2), particularly with regard to the bond performance and watertight integrity at the joint interface. To mitigate these uncertainties, the project was retrofit with a longitudinal post-tensioned detail, offsetting a majority of the tensile stresses by applying a compressive force to the transverse joint region over the pier.

In an effort to circumvent the post-tensioned detail, the Little Silver Creek utilized a much wider transverse closure over the pier. With the joint interfaces located in lower stressed regions of the deck, the transverse closure could be detailed to use conventional cast-in-place concrete materials.

The compressive components of the continuity stresses were accommodated by providing load paths between the simple span beam ends over the piers. The Keg Creek project detailed a positive connection between beam ends utilizing bolted steel tie plates. In lieu of the bolted tie plate detail, which was deemed sensitive to construction tolerances, the Little Silver Creek project detailed a fabricated steel compression block, snug fit and shimmed between beam ends. The beam ends and compression block assembly for the Little Silver Creek project would be encased within a full-depth concrete diaphragm cast integrally with the transverse deck closure using conventional concrete.

Abutments

The semi-integral abutment design utilized on the Keg Creek project featured precast concrete footing and wing elements, in combination with prefabricated backwall diaphragms cast integrally with the superstructure modules. The post-construction review identified several opportunities to improve the constructability of this design, notably citing the difficulties of precisely fitting and joining the precast abutment components (7).

The Little Silver Creek project deviated from the Keg Creek design approach, in favor of a fully integral abutment design more consistent with standard Iowa DOT design details. The design could accommodate permissible use of precast concrete footing and wing elements, but the backwall diaphragm was detailed for conventional cast-in-place construction, serving dually as the sub-to-superstructure connection and the transverse closure connecting the superstructure modules.

Piers

The Keg Creek project utilized two-column frame piers featuring precast concrete columns and precast concrete caps. The precast pier components were installed on drilled shaft foundations that were constructed outside of the existing bridge footprint prior to the ABC road closure. Assembly of the system was successfully and rapidly accomplished (7), although the weight of the precast caps essentially matched the limits of handling capacity for conventional contractors in the state.

The precast two-column frame pier system was evaluated for use on the Little Silver Creek project, but it was determined that the size of the pier caps would be too large to handle with readily available construction equipment, due to the longer span lengths and heavier design loads. Instead, the project

utilized an Iowa DOT standard pile bent pier detail, modified for increased load capacity and accommodation of permissible precast concrete pile bent caps.

FUTURE OF MODULAR CONSTRUCTION IN IOWA

The Keg Creek project demonstrated that modular PBES construction is a viable means for accomplishing ABC at the state and national level, and the Little Silver Creek project is expected to further demonstrate the versatility and adaptability of this construction method.

Iowa DOT is in the process of developing and implementing standard design details for ABC, and the working standards have already begun to feature details demonstrated successfully on the Keg Creek project. Modular construction details utilized on the Little Silver Creek project that are found to be cost-effective, durable and constructible will also be recommended for inclusion among the Department's working standards, solidifying eligibility for more widespread application on the State Highway System.

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ACCELERATED BRIDGE CONSTRUCTION IN ALASKA

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INTRODUCTION

Adoption of Accelerated Bridge Construction (ABC) approaches has been steadily increasing, but in many areas of the country the emphasis of ABC is still on minimizing traffic and mobility impacts. In many ways Alaska has always needed to consider ABC, but less as a way to mitigate traffic impacts and more due to the other inherent benefits of ABC. The Alaska Department of Transportation and Public Facilities (DOT&PF) has predominately used prefabricated bridge elements and systems and alternate contracting methods to implement ABC.

NEED FOR ABC

Unlike other states, where congestion and high traffic volumes dictate the motivation to use ABC methods, Alaska's need for ABC is based on factors such as access and climate. There are only about 1,000 state- and locally-owned bridges in Alaska, but they are spread over an area that is roughly equivalent to one-fifth of the contiguous United States. Furthermore, at least 20% of Alaska's bridges are not on the road system, which means that equipment and materials can only reach the bridge site by boat or plane. These remote bridges, and even many remote bridges on the road system, rarely have detour possibilities, which is another reason that prolonged construction is disruptive and expensive. Traffic volume is not typically a concern, since about one-third of the state's bridges have an ADT less than 100 and about 90% of the state's bridges have an ADT less than 10,000. Instead of traffic impacts, difficult access conditions challenge designers and contractors to minimize construction time which therefore minimizes costs.

Climate and daylight are the other reasons the DOT&PF encourages construction projects that can be completed in one season or less. In most areas of the state, construction activities are only practical during the summer months (beginning of May until mid-September). Summer work is often further limited by "fish windows", the environmental commitments that limit in-water work during fish-spawning times. A majority of the state's bridges cross waterways, so fish windows are a common time constraint. During the winter months (October until April) temperatures are too cold and the daylight hours are too limited to allow safe and productive construction activities. For example, the winter temperatures in Fairbanks, Alaska routinely reach -40° F and the shortest day of the year has less than 4 hours of daylight. If a bridge

project is finished in one summer, the additional time and cost for a contractor to mobilize and demobilize due to a winter shut-down are eliminated.

PREFABRICATED BRIDGE ELEMENTS AND SYSTEMS

Precast elements are a tried and true method of accelerating bridge construction in Alaska. For the many remote bridges in Alaska, especially off the road system, precast elements not only help a contractor speed construction, but also improve quality. Many of Alaska's bridges are not within adequate driving distance of a concrete batch plant, so precast elements avoid the low quality concrete that often comes with pre-batched concrete and truck mixing. Many types of precast bridge components have been historically used for Alaska's bridges, but three main components have been adopted as current standards.

Decked Bulb-Tee Girders

Alaska DOT&PF has been using decked bulb-tee girders (DBTs) since the early 1970s. The deck is an integral part of these girders, so the time for deck construction is reduced to the time it takes to weld the connecting shear tabs and grout the longitudinal shear keys.

Precast Deck Panels

Remote bridges often require the use of steel girders, which are smaller and lighter than DBTs. In remote villages, large cranes are often not available, and shipping of heavy materials and equipment is not practical or cost effective. Additionally, since cast-in-place concrete is not ideal in remote locations, precast deck panels are a more practical choice. Precast deck panels offer the same quality as DBTs and offer quicker installation than cast-in-place decks.

Pipe Pile Bent System

As a result of decades of seismic research and testing, DOT&PF has developed a pier bent system comprised of concrete-filled steel pipe piles that is standard on routine bridge piers. The hollow steel piles can be placed quickly and sometimes during the late winter months to get an early start to the construction season and avoid fish windows. In addition, the same cranes that drive the piles can often be used for placing DBTs, and thereby reduce the amount of equipment that needs to be mobilized to the site.

ALTERNATE CONTRACTING METHODS

Both design-build and construction manager/general contractor (CMGC) contracting methods have been tried in Alaska. CMGC, though limited to only one completed project thus far, has been more successful and shown more promise for Alaska's conditions and contracting climate.

Design-Build

Approximately a dozen design-build bridge projects have been contracted in Alaska. However, in many cases, the reason for selecting design-build contracting has not necessarily been to speed construction, but to encumber funding at the beginning of the project or otherwise mitigate project circumstances. While they may have accomplished other goals, the quality of bridges from design-build projects has not typically been as good as bridges from conventional bridge projects. Contractors and designers for design-build projects have often come from outside Alaska. By not being familiar with local conditions and suppliers, the construction process can often be lengthened instead of shortened.

Construction Manager/General Contractor

DOT&PF has completed one CMGC project as both the owner and engineer-of-record. Overall, the project was reported to be a success based on some of the following positive outcomes: no change orders, upfront cost savings, validation of typical DOT&PF estimates and practices, and improved coordination and relationships with stakeholders. Some reported “lessons learned” from the project were: experience with Alaskan costs is necessary, transparency in procurement is crucial, and teamwork is critical at all stages of the project.

TDOT CM/GC EXPERIENCE WITH ABC – FAST FIX 8 IN DOWNTOWN NASHVILLE, TENNESSEE

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ABSTRACT

The Tennessee Department of Transportation has embraced the Accelerated Bridge Construction and alternate project delivery philosophies in the completion of several diverse projects. The Department is now embarking on a new delivery method, Construction Manager / General Contractor on a high-profile project in downtown Nashville where four pairs of bridges and the associated segment of mainline urban interstate will be rehabilitated in only 10 weekends. The success of the project can be attributed to the collaborative efforts of the project team along with the detailed planning completed for the execution of the work during the 58 hour closure periods.

INTRODUCTION

The Tennessee Department of Transportation (TDOT) has embraced the concepts associated with Accelerated Bridge Construction in the delivery of several diverse projects. Additionally, the Departmenthas completed several successful alternate delivery projects utilizing the Design-Build delivery method.

Through guidance from the FHWA Every Day Counts Initiative, TDOT has now developed policies to allow the use of Construction Manager / General Contractor (CM/GC) as a tool to streamline project delivery. The Department has selected a high-profile project in downtown Nashville where four pairs of bridges and the associated segment of urban interstate will be rehabilitated utilizing 10 full closure weekend periods. Let's take a closer look at the path taken to allow the use of CM/GC in Tennessee.

Until recently, Tennessee statute did not allow the procurement of highway construction projects utilizing any alternate delivery methods. The design-bid-build delivery method was the only option open to the Department. In 2009, the Department worked with the General Assembly to modify procurement policy to allow for design-build project delivery and the Department has since establish policy and procedures to utilize this method on appropriate projects. The process to add CM/GC delivery method as an available option for the Department was somewhat more complicated and was only authorized by statute in 2014

thus adding another tool to TDOT's procurement strategies. The initial statute allows for three trial projects followed by an evaluation period to determine if the method should be extended, modified or sunset as an allowable delivery option. With the procurement rules established, TDOT moved forward with their initial project.

SCOPE OF THE CM/GC PROJECT

The initial scope of work to be completed on the project included the design and construction of bridge deck replacements for a pair of existing bridges on I-40 over Charlotte Avenue, bridge deck or superstructure replacement for the existing bridge pair on I-40 over Jo Johnston Avenue, and superstructure replacements for both existing bridge pairs over Herman Street / Nashville & Western RR and Clinton Street / CSXT RR using Accelerated Bridge Construction (ABC) methods. The project also include milling and resurfacing the roadway along with incidental grading, drainage, lighting and signage.

In addition, the Contractor (Kiewit Infrastructure South Co. - KISC) would be involved in the design development by providing input to TDOT and the Design Consultant (Gresham Smith & Partners – GS&P) concerning various design elements and constructability throughout the CM/GC process.

The driving issue to complete the project was the rapid deterioration of the existing bridge desks. All of the structures had exhibited some degree of deck failure, but the Charlotte Avenue bridge had three major deck failure issues over the summer of 2013 that required the closure of multiple traffic lanes and an emergency weekend closure to replace two bays of the existing deck.

Items that supported the accelerated construction schedule included the tremendous traffic load carried by I-40 which exceeded 140,000 ADT. Additionally, the central and mid-town business districts primary access points were off of this section of I-40. The area is also home to burgeoning medical campuses with five major hospitals located off of this interstate segment. Additionally, the major sport, tourist and entertainment venues are all accessed primarily from this I-40 segment. There is also currently over 20 major high-rise projects under construction in the area including residential and office space. The rapid opening of these new facilities helped develop the target schedule of having this project complete before these new buildings were completed.

Some of the specifics of the existing 1960's roadway facility that the team was to address include:

- Three lanes of original concrete pavement with asphalt shoulders in each direction;
- Narrow inside shoulders at 0.00% cross-slope with median drainage;

- Asphalt overlaid roadway with deterioration and reflecting cracking; the bridge site specifics include:
- I-40 over Herman Street / Nashville and Western Railroad which is comprised of 4 spans of continuous structural steel beams with a continuous concrete deck;
- I-40 over Clinton Street / CSXT Railroad which is comprised of 6 spans of continuous structural steel beams with a continuous concrete deck;
- I-40 over Jo Johnston Avenue which is comprised of three spans of prestressed AASHTO Beams with a continuous concrete deck;
- I-40 over Charlotte Avenue utilizing a three-span structural steel "K-Frame" design with a continuous concrete deck.

Some of the additional project challenges include:

- Project site is congested with the 4 bridges located within less than ½ mile of each other and there are frontage roads that parallel both sides of the interstate;
- Existing structures have complex geometry including horizontal and vertical curves, tapering widths and on and off ramps leading to challenging construction and fabrication;
- Bridges are actually constructed as two separate structures with a longitudinal joint located under the existing median barrier rail;
- The existing cross-slope on the bridges is substandard by current policy and contributes to some of the drainage issues along this stretch of interstate highway;
- Existing structures have been widened using different beam types and spacing than the original construction;
- CSXT railroad track is the only connection heading west out of Nashville to other regional transportation hubs. Additionally, 75% of all CSXT network traffic travels through downtown Nashville and available open track periods will be severely limited;
- Availability of work weekends will be dependent on minimizing conflicts with major downtown, university and professional sporting events;
- Five major hospitals, including the regions trauma level one and primary children's hospital all are accessed from this interstate corridor;
- Major new residential and office developments are under construction and nearing completion along this stretch of interstate leading to additional access issues for the public.

PRELIMINARY ENGINEERING STARTING POINT

Prior to transitioning the project to the CM/GC delivery method, Gresham, Smith & Partners had been working with TDOT to completed preliminary design and drawings to identified possible methods for completing the work on the bridges within an accelerated and compressed project schedule. The first two structures, comprised of continuous steel beams, have the complexity of active rail lines under the existing spans. Preliminary options were developed to utilize either full depth deck panels or superstructure units on the existing substructure elements in an effort to minimize railroad impacts.

For the crossing at Jo Johnston Avenue, the Department had done full depth deck panels on a similar interstate structure comprised of AASHTO I-Beams, so preliminary plans were developed to use the same method on this bridge.

For the Charlotte Avenue "K-Frame" structure, the replacement of the deck and maintaining the existing steel structure was seen as a viable option. TDOT had experience with full depth precast deck elements, but not on steel bridges. Adding to the challenge, the steel members on this bridge are spaced within a taper and the members are not parallel. This would complicate the fabrication and installation of the full depth deck panels. A panel layout plan was completed for this bridge. The project design team had limited further design work until the CM/GC selection process was completed.

DESIGN ALTERNATIVES STUDIED WITH CM/GC INPUT

The project team held a series of meeting to evaluate various aspects of the project. With regards to the roadway issues, the team considered the option of using precast concrete slabs to replace the existing roadway and crack and seating the existing concrete slabs and replacing with a full depth asphalt overlay.

Consideration was given to required drainage modifications, existing vertical clearance issues and issues related to constructability over a limited number of weekend closures. In the end, the team settled on removing any existing damaged concrete pavement and replacing it with full depth asphalt.

For the bridge sites, in addition to the methods identified in the preliminary engineering work, other options were presented that included:

Full superstructure replacement with prestressed concrete box beams and full depth deck panels;

- Precast superstructure units with steel beams and full depth concrete slab;
- Full span replacement with lateral slide or SPMT units;
- Elimination of spans or entire structures;

- Combination of options.

In evaluating the various options, criteria was established to develop a matrix to establish a ranking to determine which options should be carried forward for further consideration. The criteria used included:

- Duration of closure – number of impacted weekends;
- Rough order of magnitude (ROM) costs;
- Life-cycle analysis of option;
- Railroad / Utility / ROW Impacts;
- Procurement of materials and impact on overall schedule;
- Constructability;
- Risk

Clinton - Existing 6 Span Bridge - CSX RxR - Steel 36" WF Girders

Option			Cost ($M)	Cost ($M)	Total Cost	Cost	Schedule		Sch Combined	Schedule	Structure Depth	Str Depth	Maintenance	Maint	Aesthetics	Aesthetics	Total
#	Type	Type	Constr	Temp	$ Million	Rank	Procurement	Construction	(+, ✓, -)	Rank	(+, ✓, -)	Rank	(+, ✓, -)	Rank	(+, ✓, -)	Rank	Ranking
						40%				40%		10%		5%		5%	100%
1	PC	Deck Replacement - Full Dept PC Panels															
2	PC	MSE Span 1,3,4,6 45" Precast I Beam Slide Span 5, Set in place Span 2 over CSX															
3	PC	MSE Span 1,3,4,6, BT/Slide Span 1, 3															
4	PC	BT/Full Depth Panel All Spans															
5	PC	BT/Slide All Spans															
3	PC	MSE Span 1,3,4,6 36" Box with precast deck panels															
6	Stl	MSE Span1,3,4,6 36"Pl.G/Fast 14 Span 2,5															
7	Stl	36" Pl.G/Fast 14 All Spans															
8	Stl	36" Pl.G/Slide All Spans															

Legend

(+)		Most desirable
(✓)		Acceptable
(-)		Least desireable
		Lowest value for "Total Ranking" score is the preferred alternative

Sample Selection Matrix Form

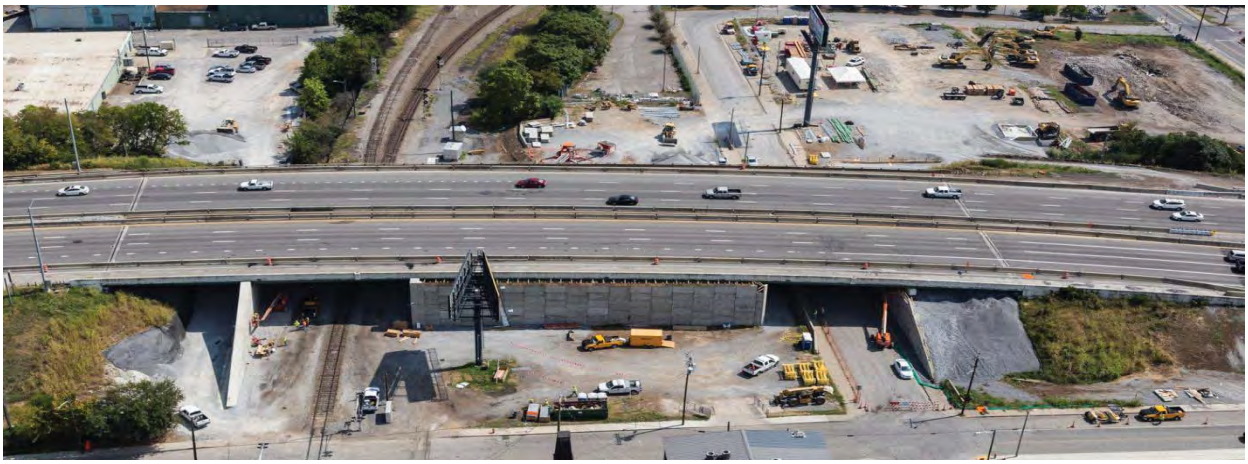
The matrix evaluation process was a valuable tool in ranking the numerous site specific options and identifying which should move forward for further consideration. One major item, material procurement, played an important role in the ultimate selection of final options for the bridge sites. In evaluating the various options for the different bridge sites, the project team had to factor in the availability of structural

steel in terms of plate availability and fabrication capacity in concert with the desired construction schedule. KISC checked with their network of suppliers and quickly realized that meeting a completion date in 2015 would be a challenge without early procurement of structural steel. This information was factored into the final selection matrix as the selection of the final site options were being finalized.

SELECTED ALTERNATIVES PER BRIDGE LOCATIONS

With the evaluation factors weighed for each location, the selected option turned out to be unique for each bridge site. For the Herman / NERR structure, it was decided to use superstructure units comprised of welded plate girders and cast-in-place deck constructed in two beam units. This option was selected based on ROW restrictions and the overall condition of the existing substructure units. The ultimate configuration of the bridges resulted in a phasing of the replacements over a two 58 hour weekend periods. Basic tasks to be accomplished on the weekends includes demolition, bearing and endwall modifications, superstructure unit placement and final paving and striping prior to opening to roadway to traffic.

For the six span Clinton Street / CSXT structure, the selected option combined span elimination and precast beams and decks panels. Spans 1, 3, 4 & 6 were eliminated and two single span bridges



Clinton Street / CSXT Structure Prior to Weekend Closure

spanned over the road and railroad. To accomplish this, cast-in-place walls that encased the existing bents on either side of the railroad were constructed. The walls had engineered backfill to be self-supporting in order to not overload the existing bents which would now act as abutments. Existing spans 3 and 4 were encased inside of a MSE wall system to support the new roadway pavement that would be installed during the weekend closures. Span 5 became a single span bridge over Clinton Street and span 6 was filled behind a MSE wall. The two single spans were constructed with precast prestressed concrete box beams, full depth prestressed deck panels and precast endwall blocks. Additionally, precast approach slabs supported by geosynthetic reinforced soil were installed at each bridge end.

The existing three-span structure at Jo Johnston Avenue was transformed into a single span structure with the elimination of the two end spans. This was accomplished by constructing MSE walls in front of the existing piers and modifying them into abutments for the single span. The backfill on these walls was again engineered to limit the load that would be applied to the existing bents. Weekend work included the construction of the single span with precast prestressed concrete box beams, full depth prestressed deck panels, precast endwall blocks and approach slabs. Additionally, the contractor had to complete the backfill of the eliminated spans and place full depth pavement.

The final bridge involved the replacement of the existing threespan “K” frame structure with a newsingle span bridge supported by a new end bent buried in engineered backfill. The existing end spans were excavated down to solid rock and the area was filled with leveling concrete to an elevation that would allow for the construction of new footings, columns and bent caps.

The site was so constrained that portions of the new cap had only one inch of clearance from the existing beams. As the substructures were built, block type MSE walls were constructed to contain the fill for the elimination of the end spans. Fill is placed as high as possible to minimize the amount of work that needs to be completed during the weekend closures.

HIGHLIGHTS OF SELECTED CONSTRUCTION ACTIVITIES

As the project shifted from design to construction, several items emerged as key factors in the overall success of the construction phase of the project. Several of those items are highlighted below.

CONSTRUCTION PHASING AND SEQUENCING

When scheduling and developing the phasing for the project, multiple factors needed to be considered in order to produce the most efficient project schedule. Each bridge location's scope of work ultimately was independent of the other structures; however, balancing resources such as labor, equipment and materials was critical in producing a realistic and accelerated schedule. The following major constraints were all evaluated in determining the sequence of construction, order of bridge weekend closures, and ultimately project completion.

- Availability and Procurement of Material – Structural steel girders, precast concrete girders, full depth prestressed concrete deck panels, and bridge bearings had critical schedules for their availability which drove the weekend closure options and final project schedule.
- Weekend Preparation Work Schedule – The construction of walls and modifications to the existing substructures at each location needed to occur prior to the weekend closure.
- Unique Equipment Availability – Cranes and SPMT availability was also critical in scheduling weekend closures.

SOUTH YARD FABRICATION AREA

The project was able to utilize existing TDOT ROW in the median of the interstate in order to set up an on-site fabrication yard for the construction of the new superstructure units for the Herman Street and Charlotte Avenue bridges. The fabrication area proved beneficial for multiple reasons:

Aerial View of the KISC South Fabrication Yard

- On-site fabrication minimized weight and width constraints of hauling superstructure units from an
- offsite location;
- Ability to match cast or preassemble structural steel, and cast-in-place concrete to improve quality and fit up for the weekend closures;
- Bridge construction near ground level in a single area instead of in an elevated situation improved efficiencies and quality and safety;

CLOSURE POUR MATERIAL

In partnership with TDOT, Irving Materials Inc., and Middle Tennessee State University Concrete Industry Management Program, a high strength ready-mix type concrete was developed that produces 3,000 PSI in 3-4 hours and at which time can be opened to traffic. The mix design was

approved for use on the project as TDOT Class X Concrete and ultimately will be available to be utilized on future accelerated bridge projects throughout the state. Developing this material eliminated the use of a bag type of grout which requires significantly more labor and hand work with variable quality results.

ELIMINATING SPANS WITH MSE WALLS

MSE walls were constructed underneath the existing bridges while still in operation in order to eliminate bridge spans. This concept ultimately reduced the amount of work that needed to occur on the weekends by making it less critical and concurrent with material procurement.

On the existing Clinton Street six-span structure, a total of four spans were eliminated ultimately making the structure feasible to construct under two weekend closure schedule. This concept was also utilized at Jo Johnston Avenue and Charlotte Avenue where the costs of wall and roadway construction were anticipated to be more cost effective than constructing additional bridge area. This also opened up additional structure type options at these locations.

WEEKEND WORK AND DEMOLITION

Weekend planning and scheduling includes the coordination of multiple subcontractors, suppliers, labor and equipment. Use of detailed hourly schedules and hour by hour snapshots of the ongoing work were tools used in order to accomplish the weekend work within the 58 hour closure window.

Every weekend starts out with the critical operation of Bridge Demolition which is scheduled to be completed within 12 hours in order to begin bridge construction activities. Bridge demolition has taken on many unique operations

where use of both cranes and excavators are used on a single bridge in order to have enough resources to remove the bridge in time.

CONCLUSION

As the project moved through the various weekends of construction towards completion, the crews were able to meet, and in most instances, beat the project requirements to have the Interstate open in time for Monday morning rush hour. The team of owner, designer and contractor worked collaboratively throughout the project utilizing the CM/GC process to deliver a project that will serve the traveling public for many years. The highlight of the project is the fact that a two to three year construction project with continuous lane closures and severe impacts to the public was completed in only 10 weekend closure periods within one construction season. The Tennessee Department of Transportation is currently working through the CM/GC process on their next project in Memphis with all expectations that is will have the same successes as the Fast Fix 8 project..

I-84 OVER MARION AVENUE, BRIDGE SUPERSTRUCTURE REPLACEMENTS USING SPMTS

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INTRODUCTION

The superstructures of Bridge Nos. 01235 and 01236, which carry Interstate 84 (I-84) eastbound and westbound over Marion Avenue in the town of Southington, Connecticut had been built in 1963 and were rated in poor condition. Each bridge carries three lanes of I-84 and due to the volume of traffic (ADT of 86,500); stage construction was not an option. New bridge superstructures were built in staging areas adjacent to I-84 in order to minimize the impact to the traveling public. The Connecticut Department of Transportation's (Department) contractor, Northern Construction Service LLC, successfully replaced the superstructures of these bridges using SPMTs during the weekend of June 27-29, 2014. The interstate was closed in both directions, traffic was routed around the site and one lane of travel was permitted to utilize the off-ramps while the existing structures were demolished and the new structures moved in utilizing SPMTs. These two new major interstate bridge structures were comprised of a composite concrete deck on pre- stressed concrete girders overlaid with bituminous concrete.

DESIGN ASPECTS

The Department hired the firm Stantec Consulting Services, Inc. to design the bridge rehabilitation. Close, Jensen & Miller, P.C. assisted the Department with project management. Stage construction maintaining two lanes of traffic was discounted due to the traffic volume. Other alternatives that were reviewed included widening or use of a temporary bridge to maintain three lanes of traffic. Both were discounted due to the cost and construction utilizing SPMTs was planned.

New England bulb tee girders were selected to replace the existing AASHTO concrete girders in order to minimize substructure work. The designer provided staging area site plans as well as plans showing suggested SPMT travel path and movement sequence. The Department offered the contractor the option of demolishing the bridges in place.

The site location afforded the Department with sufficient state owned right of way. No private properties were impacted with the exception of access during the weekend construction. Early coordination with

Utility companies identified the conflicts and necessary utility relocation was completed prior to the weekend closure.

CONSTRUCTION PROCESSES

The contractor, Northern Construction Service, LLC, elicited Marino Crane to execute the SPMT move. The contractor proposed a different sequence of construction than originally called out in the plan set. He demolished the westbound structure in place and removed the eastbound structure using SPMTs. The contractor also created two additional staging areas to facilitate the moves, erecting one superstructure over the frame of the SPMT. He excavated the staging area to a greater depth than detailed in the contract plans. He utilized an 18 inch gravel base as well as crane mats to support the temporary staging area. Steere Engineering,Inc. monitored the movement of the bridge during the weekend move. The contractor's means and methods reduced risk and made the project successful.

PUBLIC OUTREACH AND TRAFFIC COORDINATION

The Department performed extensive public outreach for this project. Department staff held a public information meeting during preliminary design following a meeting with the Southington Town Manager and Engineer. Stakeholders were informed regarding how they would be impacted by the construction activity. Department design and construction engineers held follow-up meetings with state and local emergency officials as well as local merchants.

Weekly coordination meetings were held prior to the move to discuss maintenance and protection of traffic with the contractor, traffic engineers and all emergency services personnel. The Town of Southington assisted in the public outreach by notifying residents along the local detour routes as well as sending out a reverse 911 call alerting all residents of the weekend move. The Connecticut State Police set up a command center at the nearby Interstate Rest Area to consolidate all project communication during the weekend closure. Along with the State Police, Department construction and maintenance staff, local police and fire officials reported to this command center.

During the weekend move, the Department utilized web cameras along with a dedicated webpage for the project http://www.ct.gov/dot/cwp/view.asp?a=1410&q=540374 for the public to follow the construction activity. This reduced the public activity near the project site thereby reducing job site conflicts. Because of the success of the public outreach, traffic volumes were greatly reduced during the weekend travel.

LESSONS LEARNED

The designer, contractor and construction staff held a meeting following construction to discuss process improvements for future projects. Suggestions included the following: size and quantity of bearing plate shims; advance geotechnical analysis of the staging area, maximum grade for SPMT projects, alternative superstructure types and incentives/disincentives. The Department will continue to use this technology in other locations around the state after the success of this major bridge rehabilitation project. This project as well as the majority of projects advertised by the Department utilized the design-bid-build contracting method. The Department will be looking at alternate contracting methods such as Design-Build to maximize innovation.

ACKNOWLEDGMENT

The contract documents allowed Northern Construction Service, LLC to close I-84 for up to 56 hours from 9 p.m. on Friday until 5:00 a.m. on Monday morning. They completed the project early and opened the eastbound barrel by 4:40 p.m. on Sunday and the westbound barrel by 8:15 p.m.

The Department and contractor received accolades from the Governor. He issued a press release announcing the early project completion. Here is a link to the full release:

http://portal.ct.gov/Gov-Malloy-Announces-Early-Completion-of-I-84-Bridge-Replacement-Project-in-Southington/

PHOTOGRAPHS

Both photos credit due Mr. Peter Venoutsos, Connecticut D.O.T.

BRIDGE SLIDE OVER THREE-BARREL BOX CULVERT ON DOTHAN, ALABAMA'S ROSS CLARK CIRCLE

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INTRODUCTION

The Ross Clark Circle over Beaver Creek project is an effort to construct two side-by-side bridges over an existing structurally deficient, high fill, triple-box culvert. It is part of a larger project intended to add traffic lanes to the 14 mile loop around the city of Dothan, Alabama. The accelerated bridge construction innovations employed to complete this project include structural design and construction methods.

PROJECT DESCRIPTION

The Situation

Built in the 1950s, Ross Clark Circle (RCC) provides a corridor for US-231 around the west side of Dothan. Originally intended to carry less than 5000 ADT, it currently hosts a 40,000+ ADT with a 20

year projection of nearly 73,000 vehicles. The objective for the project then is to meet this traffic need through increased capacity and subsequently reduce traffic congestion and travel time. This will be accomplished by widening the divided roadway from four lanes to six lanes. A culvert was built and 20 feet of fill was placed in order to cross Beaver Creek. This triple barrel 10’ (span) by 6’ (rise) cast-in-place box culvert is shown in the satellite photo of the roadway over the culvert above. Structurally, the culvert was designed for H-15 loads and since being opened to traffic it has seen extensive cracking to the point that it must be replaced if it is to safely carry traffic.

PROJECT CONSTRAINTS

Hydraulics

When the culvert was originally sized there was still a sizable amount of undeveloped property inside the Circle. This is significant because the stream flows from inside the Circle at this location. As the years passed residential and commercial development proceeded but without regard to runoff or detention. The consequence is a culvert that, in heavy rainfall events, has been immersed resulting in property flooding upstream. Because of the potential for scour damages resulting from allowing additional floodwater through the culvert and the litigation it would bring, it was determined that the culvert had to remain as a three-barrel 10’x6’ structure.

Utilities

As with most urban projects, utilities presented a challenge to accommodation in the design of this project. Power lines through the project area included transmission lines (115 kV) just inside the west ROW and distribution lines (7620 V) on the east ROW, in addition to other utility lines on both sets of poles. Water supply mains ran on both sides of the project, as can be seen in the site photo above. Sanitary sewer lines ran inside the east side ROW and then under the project site just before station 670+00. There are both water supply and sanitary sewer lines above ground at each side of the existing culvert.

Resistance to New Ideas

Meeting resistance from within an organization to change is nothing new and was anticipated and experienced by the ABC design team at ALDOT. When an innovative method of construction was proposed to solve the RCC issue it was not immediately well received. Several presentations had to be made not only to the ALDOT overseeing authority for the area encompassing RCC but also to Central Office construction engineers responsible for ensuring constructability and providing the special provisions necessary for the project. It took several months to explain the process and show how ABC was successfully employed in other states and could be done on this project. Although some remained skeptical, the idea was allowed.

Initial Plan

Originally the plan was to replace the culvert. Due to the water and sewer line constraints an MSE wall or retaining wall would have been required at each end to provide sufficient area for roadway widening. Additionally, because of the need for stage construction, two massive pile/drilled shaft structures would have to be installed on both sides of the culvert (approx. 42 feet clearance between them) with large whalers attached in order to hold back the 20’ fill. In addition, the pile/drilled shaft structures and whalers would have required reinstallation to accommodate the second stage culvert construction. This option was considered financially prohibitive.

Final Solution

ALDOT structural engineers inspected the culvert, finding it to be structurally deficient but still functional. They also determined that if the live load and a significant amount of dead load were removed from the culvert and cracks repaired, the culvert could continue working as a water conduit and would not need to be replaced. It became apparent that the best solution to these problems was to build two single-span bridges over the existing culvert.

THE ABC CHALLENGE

In order to maintain roadway elevation over Beaver Creek, the original plan for RCC was to build a culvert and bring in fill. The embankments on both sides were placed at a 1½:1 backslope.

For the two bridges over the culvert, the overseeing authority did not want to use an on-site detour or traffic diversion due to high traffic volume, the potential for traffic stopping accidents, and the loss of working area for the contractor. The offsite detour, though short, would take traffic through residential areas on a two-lane road. This was considered a very unattractive proposition for the six months it would require to build a bridge conventionally.

The solution then was to apply ABC innovation and look at building the bridges adjacent to the roadways and then slide or roll them into place. Doing so would require special attention to the three issues that varied from conventional construction and would require development of new special provisions.

Falsework Supporting Superstructure

Although falsework is regularly used on ALDOT construction projects, it had yet to be used to support large sections or all of a bridge superstructure. However, since the principles remained the same this issue was handled with a special provision and review of the contractor's engineer's calculations and working drawings.

Build Abutments under Traffic

On many ABC bridge replacement projects new abutments are built inside the existing ones, resulting in a shorter bridge. However, in this case there are no existing abutments. They had to be built under traffic. ALDOT engineers studied other state's experiences with this and found a similar project completed by the Colorado DOT over Wray River on SR-34 in 2012. The primary differences are traffic volume (1800 ADT vs. 40,000+ ADT) and abutment cap size and length. Because the abutment shoring is temporary the design of the structure was left to the contractor. ALDOT engineers, rather than trying to solve the shoring design issue, decided it would be best to leave that to contractor ingenuity – albeit with his engineer's design according to the LRFD Bridge Code.

Bridge Slide / Roll

In some respects this was considered by ALDOT design engineers to be the easiest aspect of the project. Not only were there many examples of this being done elsewhere in the country, but many of Alabama's bridge contractors had experience with bridge raising – a vertical application of the same principles involved with bridge sliding. Iowa's DOT provided us with slide special provisions which were tweaked and adapted to the Dothan project.

The Pièce de Résistance

Of the three new ABC considerations taken into account to accomplish this project, we see the successful installation and use of the abutment shoring to be at least as significant as the slide on this project. Although this is our first bridge slide and that in in itself is important, this project pushes the known limit on building substructure elements under traffic. To date, both abutments have been built this way – one is currently buried and the other still sits under cover of shoring boxes. We believe this accomplishment is something other states ought to be able to reference for bridge projects where traffic cannot be moved, a shorter bridge is unacceptable, and construction must be achieved under traffic.

FDOT PRECAST BENT CAP DEVELOPMENT AND IMPLEMENTATION

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ABSTRACT:

This paper presents an overview of the development of a PBES-ABC Precast Intermediate Bent Cap standard based on a recent pilot project featured in a presentation at the 2014 National ABC Conference. A new FDOT Developmental Design Standard for intermediate pile bent caps and multi-column piers provides connection details refined during the pilot project. Details for the Pile Bent Caps also incorporate some of the work from a 1996 preliminary study under FDOT Project No. 510703 improvements based on the first generation TxDOT standards for Prestressed Concrete Piles. Details for the Multi-Column Pier Cap are similar to the SHRP2 Project R04-RR1 (Appendix E) ABC Standard Plans, except the connection design utilizes the recommendations from NCHRP Report 681 with the alternate details for regions of low seismicity. A modified version of the FDOT's Pile Bent Mathcad program has also been developed as a design tool to assist in completing the necessary Contract Documents for streamlining implementation on Department projects.

INTRODUCTION:

Precast bent caps have been identified by the FDOT as a cost effective means of implementing one practice of Accelerated Bridge Construction (ABC) under the FHWA ***Everyday Counts*** initiative using Prefabricated Bridge Elements and Systems (PBES). This type of element has been used in previous FDOT projects and other states with project specific designs and connection details. To ensure consistent design assumptions, leverage economy of scale from standardized components, minimize the perceived risk and develop Contractor expertise, standardized details for Precast Intermediate Bent Caps have been developed for use in Florida.

Details for the Intermediate Pile Bent Caps incorporate some of the work from a 1996 preliminary study under FDOT Project No. 510703 (1), and improvements based on the first generation Texas DOT standards for Precast Concrete Bent Cap Options (2)(3). Development of a Steel Pipe Pile and H-Pile option are also planned for 2016. A grout rheology study (4) is currently underway to assess the potential problems with installation tolerances and grout sensitivity to temperature and flow rate for pile pocket

connections. Details for the Multi-Column Pier Caps are similar to the SHRP2 Project R04-RR-1 (5) except the connection design utilizes the recommendations from NCHRP Report 681 (6) with the alternate details for regions of low seismicity. Simple enhancement of the Multi-Column Pier Cap utilizing narrow spaced twin-columns could also provide an economical alternate to cast-in-place Hammerhead piers.

A modified version of the FDOT's Pile Bent Mathcad program (7) has been developed as a design tool to assist in completing the necessary Contract Documents for streamlining implementation on the Department's projects, and providing a framework for future enhancements.

BACKGROUND:

FDOT has previous experience with precast substructure components on various projects over the last 30 years. This does not include the extensive use of pretensioned concrete piles, which have been used in Florida since the 1950's (8) and precast concrete piles starting in the 1920's. More ambitious was the incorporation of: Precast post-tensioned hollow column units on Seven Mile, Channel 5 and Sunshine Skyway bridges (9) in the 1980's; Precast I-shape columns and U-shaped caps on the US41 Edison Bridge (1992) (10); and precast bent caps on prestressed piles for the I-275 Henry H. Buckman Bridge (1996). Walt Disney World's Reedy Creek Bridge was another project completed in Florida in 1997 utilizing precast bent caps (11), although not under the jurisdiction of the FDOT.

Photo 1: Edison Bridge Precast Bents (FDOT 1991)

Photo 2: I-275 Henry H. Buckman Bridge Precast Bent Cap (FDOT 1996)

The State Structures Design Office commissioned an effort to standardize precast bent caps in the mid-1990's under Project No. 510703 (1). The initial standard drawings developed from this project were never implemented for undisclosed reasons. These efforts were motivated by the successful use of precast substructures on the previously mentioned projects in the 1990's. Since this time several other FDOT

projects have successfully incorporated precast bent cap elements, including: SR 300 St George Island Bridge (2004) (12), and US 90 over Little River Bridge (2014) (13).

There have also been several recent national and state PBES substructure initiatives, including: NCHRP Report 681 - Development of a Precast Bent Cap System for Seismic Regions (8); SHRP2 Report S2-R04-RR-1 Standard ABC Plans (14); and Texas DOT standards for Precast Concrete Bent Caps (2)(3)(15). The most recent FDOT experience is from a pilot project involving the design and construction of four replacement highway bridges on US 90 over Little River and Hurricane Creek, which was presented at the 2014 National ABC Conference (13). By utilizing precast components the FDOT gained valuable knowledge about construction techniques, durability concerns, and cost for precast bridges in a low risk application. Positive project results led to advancing Precast Intermediate Bent Caps as the first ABC-PBES element to be fully standardized in Florida.

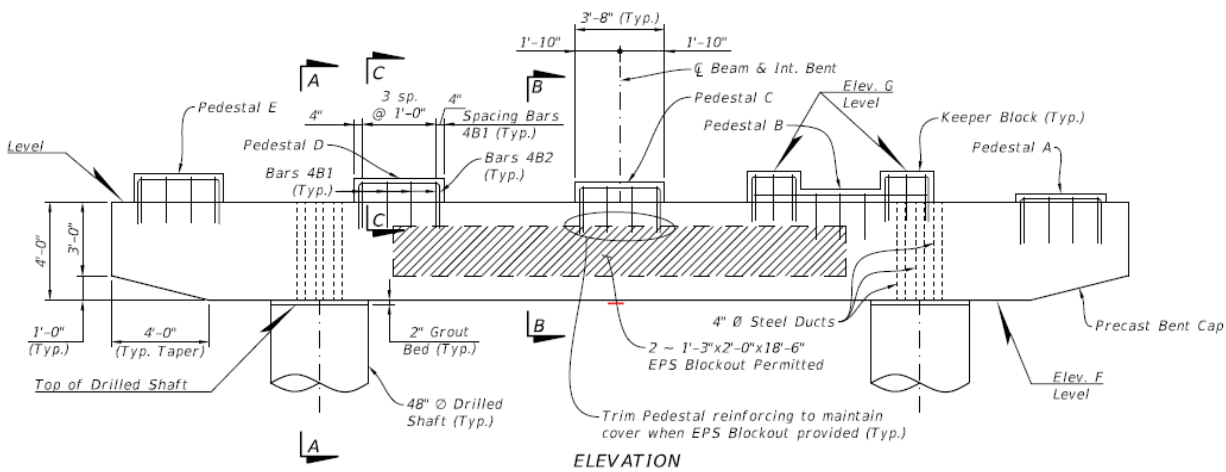


Figure 1: Precast Intermediate Bent Cap from US 90 over Little River pilot project

STANDARD DEVELOPMENT

The new FDOT Developmental Design Standard (D20700 series) will provide connection details refined during the US 90 pilot project, which were primarily based on NCHRP Report 681, while also taking into consideration general details from the ABC Standard Plans developed under the SHRP2 Project R04-RR-1. The reinforcing and cap dimensions will not be predesign as was developed for the ABC Standard Plans, but rather incorporate standardize geometry, details, connections and reinforcing configurations to assist the designer in completing project specific designs. This concept is similar to that used for FDOT's prestressed beam standards where the designer must complete a table of standard variables which the Contractor's precaster can directly build from or develop shop drawings if requested.

The strategy was to develop three levels of aesthetics and economy as depicted in *Figure 2* and described below:

Level 1 – Pile Bent Caps, are prismatic members with simple recessed pile pockets, usually designed as pinned or partially restrained connections (see *Figure 2a*).

Level 2 – Multi-Column Pier Caps, typically utilize two or three columns (round or square) with dowels in grouted ducts for partial or full-moment connections. The cap geometry is very similar to the SHRP2 R04-RR-1 ABC Standard Plans with tapered ends on rectangular caps (see *Figure 2b*).

Level 3 – Hammerhead Cap on Twin-Column Pier, is intended for enhanced aesthetics or improved horizontal clearance above congested roadway alignments. The caps utilize taper rates similar to the Level 2 caps, but with much longer cantilevers and closely spaced twin-columns or either round, elliptical or rectangular cross section (see *Figure 2c*).

All options can use enhanced aesthetic treatments with simple form inserts to create shadow lines or contextual shapes, and/or textured surface treatments with inexpensive form liners (see *Figure 2d*).

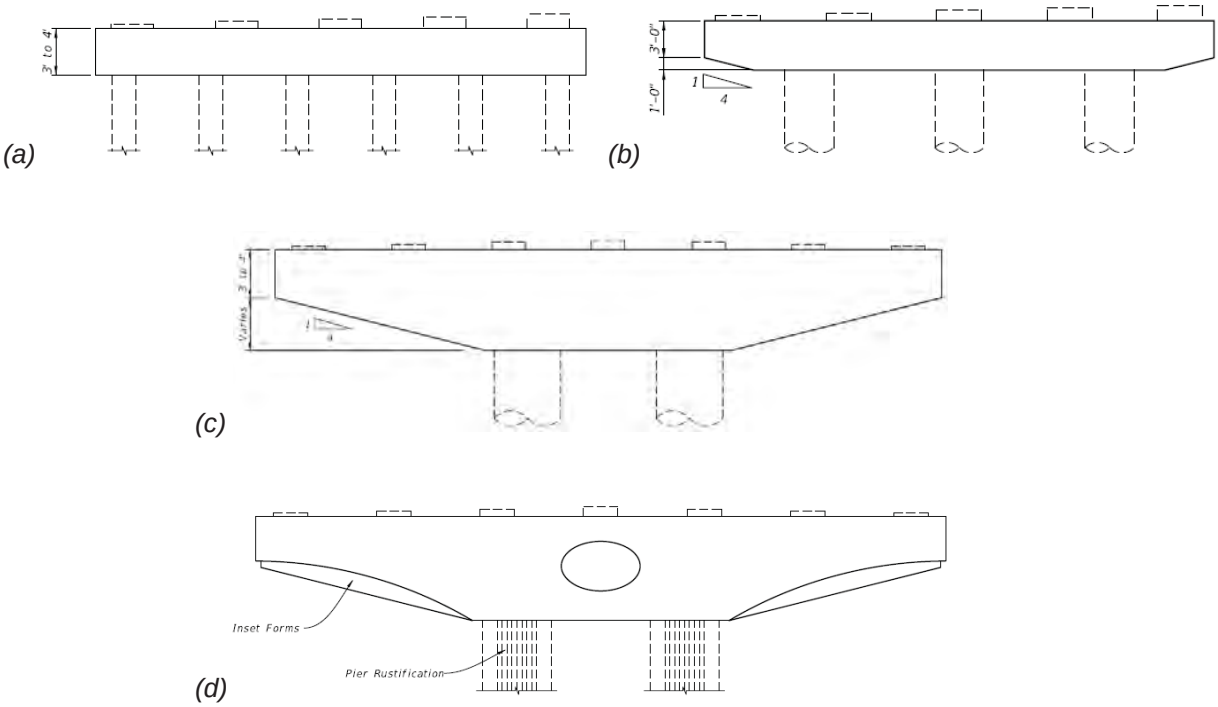


Figure 2: Precast Cap Options (a) Level 1 – Precast Pile Bent Caps; (b) Multi-Column Pier Caps; (c) Hammerhead Cap on Twin-Column Pier; (d) Potential aesthetic enhancements

The modified version of the FDOT's Pile Bent Mathcad program is currently under final QC testing and will provide a necessary design tool to assist in completing the design and construction plan data tables. Comparison with two designs recently completed in-house (see *Figure 3*), a published TxDOT Pile Bent Design Example (June 2010), the SHRP2 R04-RR-1 two-column bent cap design example (see *Figure 4*), and analysis with Bentley's RC Pier software show good correlation of results. Deviations in the results can be explained by the refinements in modeling and loading assumptions for the different designs.

US90 Project: 2-Drilled Shaft Cap Design	Str_I $+M_u$ (kip*ft)	Str_I $-M_u$ (kip*ft)	V_u at Int. face of Ext. Col. (kip)	$+M_r$ w/ 18#11 (kip*ft)	$-M_r$ w/ 20#11 (kip*ft)	V_r w/ #5@6" (4 legs) (kip)
EOR's Design	2988	-3255	598	4818	5298	719
FDOT Mathcad (conc.)	3196	-3286	895	4825	5274	687
FDOT Mathcad (distr.)	2947	-3286	798			
Difference (distributed)	1.4%	-0.9%	-25.1%	-0.1%	0.5%	4.7%
RC Pier	3471	-2874	773	4891	4710	715
RC Pier vs. Mathcad Diff.	17.8%	-12.5%	-3.1%	1.4%	-10.7%	4.1%

US90 Project: 6 x Pile Bent Cap	Str_I $+M_u$ (kip*ft)	Str_I $-M_u$ (kip*ft)	Vu (kip)	$+M_r$ w/ 7#9 (kip*ft)	$-M_r$ w/ 8#6 (kip*ft)	V_r w/ #5@7.5" (2 legs) (kip)
EOR's Design	1255	-520	330	1348	921	365
FDOT Mathcad (conc.)	981	-443	467	1347	924	458
FDOT Mathcad (distr.)	753	-421	322			
Difference (distributed)	-21.8%	-14.8%	-2.4%	-0.1%	0.3%	25.5%
RC Pier	590	-495	412	1160	959	453
RC Pier vs. Mathcad Diff.	-21.6%	17.5%	28.1%	-13.9%	3.8%	-1.0%

Figure 3: Comparisons of US 90 pilot project designs with new FDOT Mathcad program.

SHRP2 Example 3b: 2-Column Cap Design	Str_I +M_u (kip*ft)	Str_I -M_u (kip*ft)	Vu (kip)	+M_r w/ 10#11 (2 rows) (kip*ft)	-M_r w/ 8#11 (kip*ft)	V_r w/ #6@ 9" (4 legs) (kip)
SHRP2 Example 3b	1901	-2263	354	2823	2396	809
FDOT Matchcad (dist.)	2626	-1799	351	2823	2396	711
Difference	38.2%	-20.5%	-0.9%	0.0%	0.0%	-12.1%
RC Pier	2504	-1613	276	2802	2422	663
RC Pier vs. Mathcad Diff.	-4.6%	-10.3%	-21.3%	-0.7%	1.1%	-6.8%

TxDOT LRFD Example: 3-Column Cap Design	StrI +Mu (kip*ft)	StrI -Mu (kip*ft)	V_u at Int. face of Ext. Col. (kip)	+M_r w/ 7#11 (kip*ft)	-M_r w/ 7#11 (kip*ft)	V_r w/ #5@8.5" (kip)
TxDOT Example	1190	-1325	354	1778	1778	376
FDOT Matchcad	1437	-1099	351	1778	1778	307
TxDOT vs. Mathcad Diff.	20.7%	-17.0%	-0.9%	0.0%	0.0%	-18.5%
RC Pier	1289	-1047	326	1794	1794	314
RC Pier vs. Mathcad Diff.	-10.3%	-4.7%	-7.0%	0.9%	0.9%	2.4%

Figure 4: Comparisons of other design examples with new FDOT Mathcad program

The Developmental Design Standard will be refined over the next several years based on the ongoing US 90 pilot project monitoring and several small scale projects, with possible incorporation into the predesigned Off-System Bridge Standards (Index D30000 series) currently under in-house development. Similar to the Buckman and St George Island bridges, several long bridge projects are scheduled to begin design within the next 5 years where this standardized application could provide significant time and cost savings. Some of these potential projects include the new westbound 118th Ave Viaduct East of 40th St (1,600 ft.) (17), Anna Maria Island Bridge replacement (3,500 ft) (18), and Howard Frankland Northbound Bridge replacement (16,000 ft.) (19) all located in the Tampa Bay area; and Pensacola Bay Bridge replacement (16,000 ft.) (20) in the Florida panhandle.

Figure 5: PD&E concept for Pensacola Bay Bridge

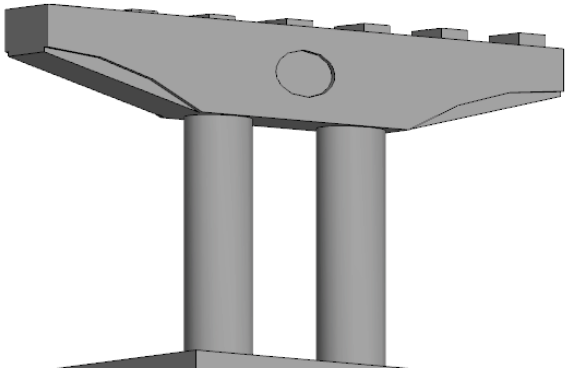
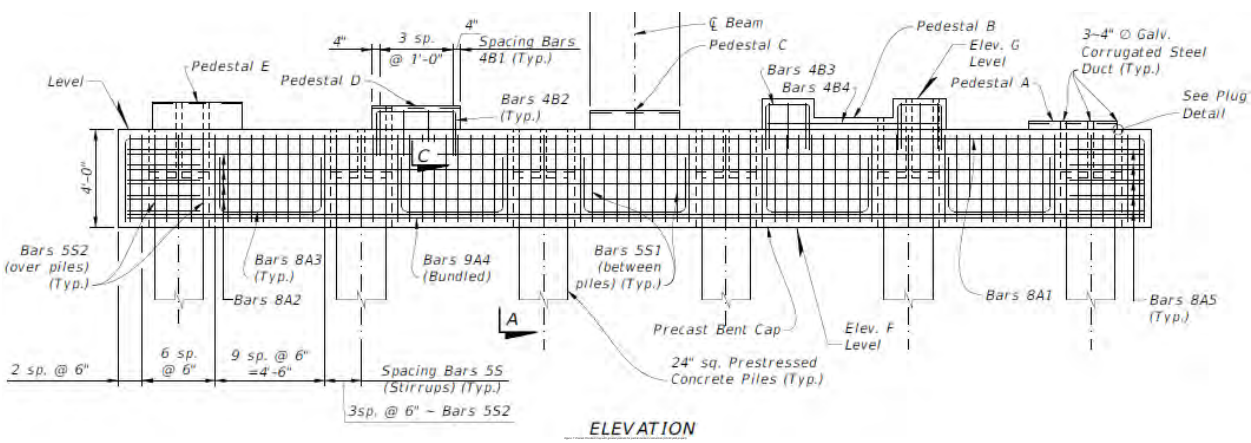


Figure 6: Proposed PBES Twin-Column Pier with enhanced aesthetic treatments

Substructure cost savings are expected for short and medium length bridges once contractors become more familiar with PBES techniques and precaster's can confidently amortized forming costs. Both of these improvements will be facilitated by FDOT's commitment to this type of ABC practice through standardization. Immediate cost savings are expected for water crossings and long bridges with lower effective labor rates due to reduced working time over water, repetitive construction, and economy of scale with precast components. Estimated construction times could be reduced by 15-30 days for each bent, especially if precast beam seats are utilized.

PBES DETAILS

The Intermediate Pile Bent Caps incorporate some of the work from a 1996 preliminary study under FDOT Project No. 510703, improvements based on the first generation TxDOT standards for Precast Concrete Bent Cap, and details from the US 90 pilot project 90% Design Plans (Pile Alternate AA2). Future development of a Steel Pipe Pile and H-Pile option, and a Hammerhead Cap with a twin-column pier planned for 2016. The grouted pile pockets with 12-inch embedment are typically assumed to function as pinned connections. When lateral deflections need to be minimized, partial moment connections can be designed using deeper pile head embedment (see *Figure 7*) with capacities based on Harries and Petrou (16).



The Multi-Column Pier Cap connections to the columns were based on dowel bars grouted in corrugated metal ducts described in NCHRP Report 681 (6). Due to Florida's location in a low-seismicity zone, the simplified grouted duct connections discussed in Chapter 3 and shown in Figures 8 & 9 are utilized. These types of connections provide more construction tolerance than the proprietary grouted bar-splice couplers used previously on the Edison Bridge and shown in the ABC Standard Plans. These simplified details are usually sufficient for most designs in Florida, except where design for vessel collision requires a full moment connection to accommodate plastic hinging. Additionally, grouted duct connections do not

require exclusive proprietary grouts or pressure injection to secure the dowelled connection, since they can be gravity fed using a number of commercially available fluid (precision) grouts. Based on the experience from the US 90 pilot project, the spiral confinement reinforcing around the grouted ducts may be substituted with #4 hoop bars and designs with more than two legs of vertical shear reinforcing per cross section, will utilize single bars with 135 degree and 90 degree hooks at opposite ends for each leg. Both these modifications will provide maximum flexibility for laying out the longitudinal reinforcing, assembling the complete reinforcing cage and repositioning grout ducts if field adjustments become necessary.

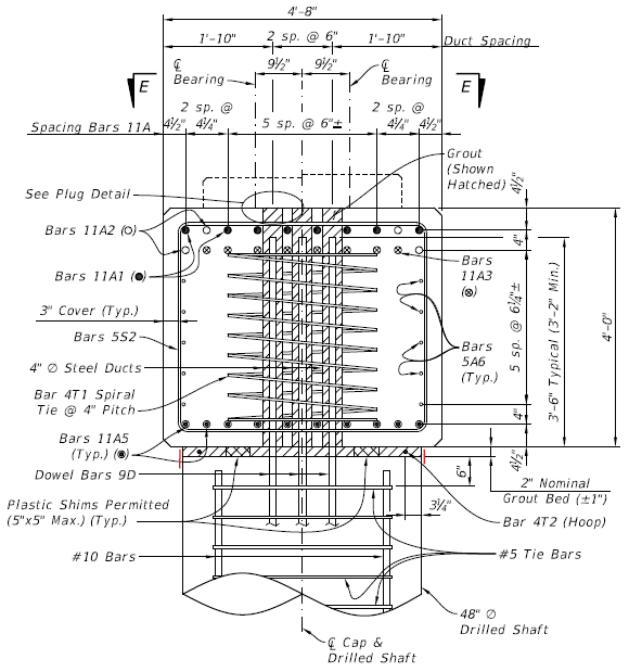


Figure 8: Typical section of precast intermediate bent cap at grouted duct connection (US 90)

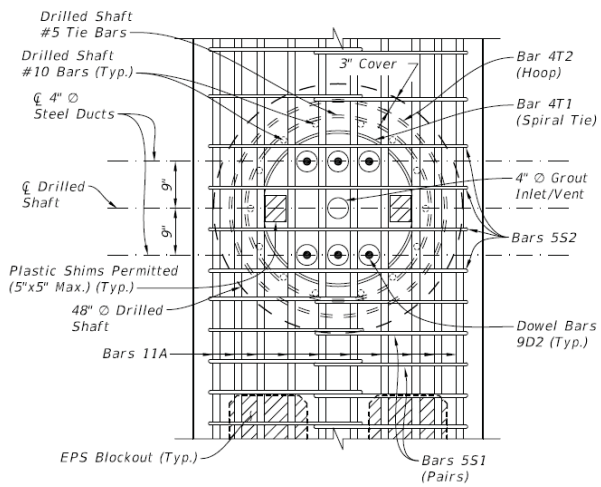


Figure 9: Plan view of precast intermediate bent cap at grouted duct connection (US 90)

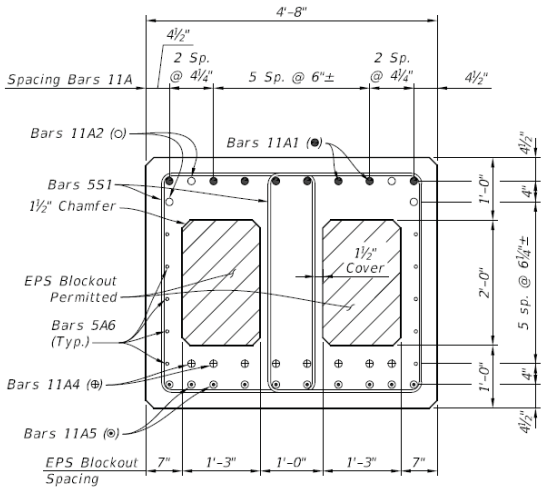


Figure 10: Precast intermediate bent cap voided section (US 90 pilot project)

Photo 3: EPS voids prior to concrete placement (US 90 pilot project)

In order to reduce the precast cap weight, the use of optional expanded polystyrene (EPS) voids are permitted. Concrete cover to the reinforcing adjacent to the EPS voids was detailed at 1.5-inches for the US 90 pilot project, however FDOT recent policy (21) established 3-inches minimum cover for internal surfaces in all environments. Resolution of this issue is important since additional cover greater than the ***AASHTO-LRFD*** 1-inch minimum, disproportionately affects the lifting weight with arguably minimal durability benefits. The negative effect on lifting weight is especially significant for multiple cell voids similar to those shown in *Figure 10*. The use of voided precast units also has the benefit of reducing the concrete volume to surface area ratio mitigating the potential for mass concrete thermal cracking.

The grouting specifications for these standards were developed using the NCHRP Report 681 Attachment CS with additional modifications recommended by the FDOT State Materials Office. The standard pre-approved non-shrink grouts on the FDOT Approved Products List (APL) do not meet the minimum desired specification requirements, which will initially necessitate the implementation of a Modified Special Provision (MSP 934) (22). Project specific approval of a non-APL grout may lead to significant delays, therefore pre-approving grouts is preferred to expedite construction especially on more time sensitive installations. Mock-up grouting tests are recommended to familiarize the Contractor with the grouting process prior to beginning construction of the connections.

Templates will be required in the top of the cast-in-place columns to ensure the connection dowel bars are positioned within the construction tolerances. On the US 90 pilot project the contractor's initial template was not sufficient to secure the dowels during the column concrete placement on the first intermediate bent. The template design was modified to provide a more rigid system for the remaining bents.

Fortunately not all the precast bent caps had been cast, so the corrugated metal ducts could be easily repositioned to accommodate the as-built location of the column dowel bars. This highlights the advantage of using uniform cap geometry and not precasting beam seat pedestals with the caps.

Fabrication tolerances have been specified and clarified on a separate detail sheet (see *Figure 11*), similar to the ABC Standard Plans.

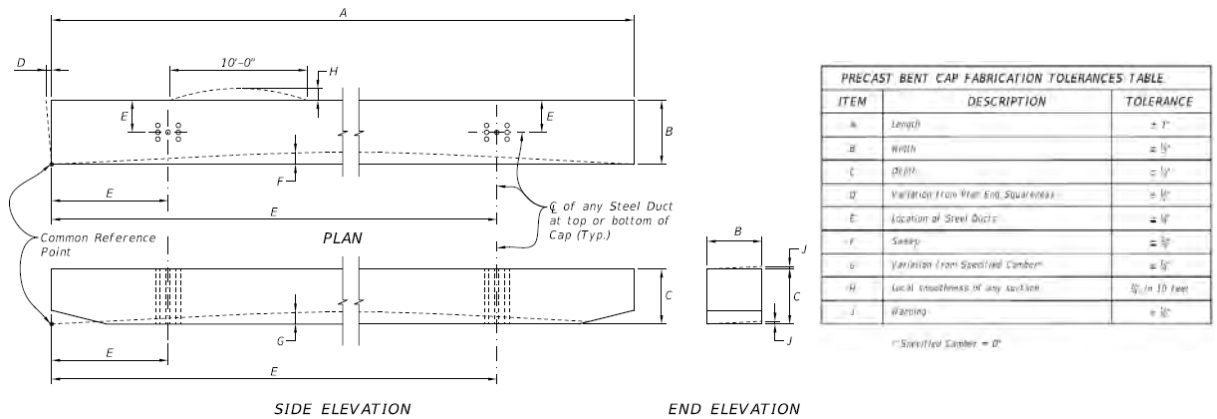


Figure 11: Standard Fabrication Tolerances (Multi-column Pier Cap)

The longitudinal flexural reinforcing is usually significantly heavier for FDOT bent cap designs compared with other states. This is primarily due to the FDOT mandated Service III design check limiting the reinforcing tensile stress to 24 ksi (23) which usually controls the design (see *Figure 12)*. The Service III requirement is intended to minimize in-service cracking and enhance durability but has been found by the author to provide inconsistent results for elements with different depths and concrete cover (24). It is proposed that the Service III criteria be modified in accordance with the commentary in ***AASHTO-BDS LRFD*** 5.7.3.4 by using a lower exposure factor to control crack size rather than a prescriptive stress limit on reinforcing.

				STRENGTH I		SERVICE I			SERVICE III (*FDOT-SDG 3.10*)		
Design Example: 70' Hammerhead Cap with Twin-Column Pier	**BAR SIZE** (#)	**MINIMUM No. OF BARS REQUIRED** 1st layer	2nd layer	**Str I -M_u Check**	**Str I +M_u Check**	**Max. Bar Spa.** (in.)	**Max Crack Width (Serv I top) *** (in.)	**Check (Spa./ CW)**	**Max. f_s (Serv III top)** (ksi)	**Max. Crack Width (Serv III top) *** (in.)	**Check ($f_s \leq 24$ ksi)**
STRENGTH I	11	19	19	OK	OK	8.5	0.013	OK	38.1	0.013	NG
SERVICE III (Limit 24 ksi)	11	31	31	OK	OK	18.3	0.008	OK	23.9	0.007	OK
SERVICE I (Exposure factor = 1.00)	11	16	16	OK	NG	6.0	0.017	OK	45.0	0.016	NG
SERVICE I (Exposure factor = 0.60)	11	23	23	OK	OK	3.9 [Δ]	0.011	OK	31.8	0.010	OK

** Estimated crack width based on Frosch Equation: $w = 2(f_s/Es)*\beta_s*\text{sqrt}[(dc)^2+(s/2)^2]$*

[Δ] This value is 11.8 in. when the Exposure factor = 1.00.

Figure 12: Minimum longitudinal flexural reinforcing required to satisfy different limit states

The precast bent caps are detailed leveled on the top and bottom with stepped height beam seat pedestals to accommodate cross slopes (typically 2%) and variable beam grades as shown in *Figures 1 & 7*. Designing bent caps leveled simplifies the column or pile connection details, which reduces another potential source of error during construction. While it is possible to precast the beam pedestals with the rest of the cap, the recommendation is to require these be cast-in-place in the field because adjustment to some beam seat elevations are inevitably required. C-I-P beam seat pedestals also have the advantage of reducing the cap lifting weight, mitigating conflicts with duct grouting, facilitating greater cap placement tolerances, accommodating deviations from the predicted beam cambers, and providing a more robust seal over the tops of the grouted duct connections and lifting points.

The standard designs for the precast bent caps deliberately do not require any pre-tensioning or post-tensioning. This will simplify on-site or near-site fabrication when desired by the contractor. Industry certification and FDOT approval of a temporary precast facility is much more feasible and less expensive than for a temporary prestressing facility.

There will be no specific restriction on precasting end bents, but typically contractors do not pursued this type of contract change when cast-in-place details are shown in the Plans, possibly due to unfamiliarity with PBES-ABC techniques and/or concern for potential schedule delays waiting on design approvals. On future PBES projects, the FDOT may include both precast and C-I-P alternatives for the end bents to better identify the lowest cost solution. In the author's opinion, the end bents should match the intermediate bent geometric configuration as much as practicable to maximize the potential for reuse of the precaster's forms and provide the most economic benefit. C-I-P backwalls and wingwalls may be the optimal solution when construction of the end bents is not scheduled on the critical path.

DEVELOPMENTAL STANDARDS FAMILY:

Index D20700 series will be broken into sub-groupings to facilitate future development and expansion of substructure PBES standards with different supporting components and connections. The proposed numbering and grouping is show below:

Level 1:

D20700 – Precast Intermediate Bent Cap Notes and Fabrication Tolerances

D20702 – Precast Intermediate Bent Cap Connection Details

D20703 – Precast Intermediate Bent Cap Details – with Prestressed Concrete Square Piles

D20704 – Precast Intermediate Bent Cap Details – with Prestressed Concrete Cylinder Piles

D20705 – Precast Intermediate Bent Cap Details – with Steel Pipe Piles

D20706 – Precast Intermediate Bent Cap Details – with Steel H-Piles

Level 2:

D20720 – Precast Multi-Column Pier Cap Notes and Fabrication Tolerances

D20722 – Precast Multi-Column Pier Cap Connection Details

D20723 – Precast Multi-Column Pier Cap Details – with C.I.P Columns or Drilled Shafts

D20724 – Precast Multi-Column Pier Cap Details – with Prestressed Cylinder Concrete Piles

D20725 – Precast Multi-Column Pier Cap Details – with Precast Rectangular Columns

Level 3:

D20740 - Precast Hammerhead Pier Cap Notes and Fabrication Tolerances

D20742 – Precast Hammerhead Pier Cap Connection Details

D20743 – Precast Hammerhead Pier Cap Details – with Precast Rectangular Columns

CONCLUSIONS

Construction is 100% completed on the US 90 pilot project substructures with positive feedback from the Contractor and Construction & Engineering Inspectors. The grouted duct connections are very promising based on the grouting mock-up tests and the six as-built precast intermediate bent caps. This type of detail appears to be very forgiving and relatively easy to fabricate with the only downside seaming to be the difficulty in producing grout with an optimal flow rate during hot weather. The FDOT is pursuing further research into the grout rheology, with full deployment of the precast intermediate bent cap standards under the D20700 series scheduled for 2016.

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Rafael Kampmann and Dr. Michelle Roddenberry from the FAMU-FSU College of Engineering lead the grout rheology testing and US 90 monitoring team respectively, and provided additional construction photographs and project information. Will Potter and David Wagner at FDOT's Marcus H. Ansley Structures Research Center are currently assisting in the grout rheology study.

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SYSTEMATIC DEVELOPMENT, IMPLEMENTATION, AND DEPLOYMENT OF

PRE-FABRICATED BRIDGE PIER

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This presentation illustrates Wisconsin Department of Transportation's (WisDOT) approach and experience in the development of policy, standards, details, and provisions for Precast Piers as a sub-set of Prefabricated Bridge Elements and Systems (PBES). WisDOT has employed a systematic process of research, pilot projects, debriefings, lessons learned, and cyclical updates to details and specifications. The results of this systematic approach represent progressively improved generations of PBES/ABC technologies that better achieve project goals. Through WisDOT's participation in the Strategic Highway Research Program 2 (SHRP2 - R04), we have had the opportunity to develop and implement pre-cast piers as a Lead Adaptor. Through development, WisDOT has learned a number of important lessons that we believe will benefit other DOT's as they implement PBES in their projects. Some of these lessons include standardization of pier geometry, simplification of fabrication and forming, and interchangeability with conventional cast-in-place pier construction. This presentation will present specific challenges and solutions that WisDOT has faced in the implementation of our precast pier elements.

RESEARCH

We have utilized a number of programs and initiatives to explore and develop ABC technologies. The FHWA's Innovative Bridge Research and Construction (IBRC), and Innovative Bridge Research and Deployment (IBRD) programs, and our own Wisconsin Highway Research Program (WHRP). Beginning in 2008, WisDOT began development on precast piers and abutment projects to explore this technology. These projects include the use of precast post-tensioned deck panels, precast pile supported abutments, and precast open bent pier caps. Some of the initial lessons learned from these first applications related to issues with weight of the elements, construction tolerances, and requirements for transport and erecting of the elements. We were successful in the design and construction of these initial projects; however, comments from industry (fabricators and contracts) were not favorable to the use of these elements. Given the concerns, issues and costs of these initial applications at the time, the department was not in a hurry to

deploy precast elements until some of these issues could be resolved. Through our Wisconsin Highway Research Program (WHRP), we were able to approach some monitoring and documentation of performance of the experimental technology. With the information and experience from our initial research applications of these technologies, we were able to move forward with project specific implementations that addressed project specific goals.

CUSTOM APPLICATIONS

One of our recent experience with custom applications was the 2013 construction of the Rawson Avenue (Milwaukee, WI) overpass as part of the IH 94 North-South project. The project consisted of the reconstruction of a full diamond interchange over IH-94. The need for ABC construction was driven by the need to promote safety and address impacts to the public. The ADT of IH-94 is well over 100,000 ADT and Rawson Avenue is 10,000 ADT. The additional user costs for Rawson Avenue alone were calculated to be over one million dollars with the use of conventional construction process. The ABC application on this project included pre-installation of pile sleeved for staged MSE Wall Abutments, Precast Piers, and Self Propelled Modular Transport (SPMT) movement of 2 Spans. Precast piers were chosen to minimize the impact to traffic in the lanes adjacent to the pier construction. By the use of the precast piers, the number and durations of the lane drops were greatly reduced. The use of the precast piers also saved 3 to 4 weeks from the construction schedule.

The pier configurations reflected the corridor aesthetic theme that incorporated many angular transitions into the columns and cap elements.These unique aesthetic features along with the tolerances requires for the grouted coupler connectors made fabrication challenging and costly. The cap element weights up to 45 Tons made lifting in the fabrication plant challenging as well. The project proceeded and the precast piers were installed successfully and help meet the goals of safety and impact to the public.

Lessons were realized during fabrication and erecting that included the need to grind beam seats due to meeting construction bearing elevations. Also identified during the post construction debriefing was the option and the advantages of using bar couplers up to two sizes greater than the connected bars to allow more tolerance during fabrication and erecting.

STANDARDIZATION

In 2013 we applied for and received a grant from FHWA through the Strategic Highway Research Program (SHRP2 (R04)) as a Lead Adaptor for precast piers. We were under design of our IH 39 corridor. This expansion project from the Illinois State line to Madison WI provided the opportunity to integrate PBES on a larger scale and strive for economy of repetition. There were over 60 bridge sites

that would be candidates for the use of the precast pier elements. Our approach was to work with the IH 39 Corridor Team to develop a set of standards that could be used for the construction of standardized piers along the corridor. Again, this provided the opportunity to mitigate safety issues and impacts to traffic operations that would be related to conventional pier construction adjacent to live traffic lanes.

We drew upon the lessons learned from our previous precast efforts and developed a series of pier standards that reflected more easily formed elements with more uniform shapes and cross section. One of the goals that we had was to encourage industry to invest in re-usable forms for the standardized column and cap elements. With standardized reusable forms, we should realize a project level cost savings as a result of the capital cost being distributed over a number of projects. Even though we had significantly streamlined the cap elements, the fabricator did indicate that the casting of the beam seats on the caps were challenging. Also the variable depth of the cap due to the beam seats caused the caps to sit have excessive weight that provided challenges during lifting in the fabrication shop as well as the challenge to get enough wheels under the caps during transport to make it a legal load. In 2014, during the first installation of this new generation of precast elements on our IH 39 corridor, the fabricator and contractor indicated that fabrication and installation overall went smoothly.

SIMPLIFICATION & INTERCHANGEABILITY

To promote economy and ease of deployment of the precast elements, we created a new generation of standards that reflected the lessons learned in fabrication and erecting on the projects to date. One of the identified goals was simplification of the pier cap element fabrication. To date, the casting of the caps and beam seats had shown to be difficult forming with risks associated with forming/casting errors and potential need to grind down the beam seats. To address this issue, we made the cap elements of uniform cross section. To provide for the beam seat elevation variations, we incorporated precast variable depth bearing blocks that would be affixed to the top of the pier caps during erecting that would provide the needed individual beam seat elevations and slopes. There are multiple benefits to this approach. First, forming and fabrication of the elements is greatly simplified. Fabricators could develop and have multiple reuse of cap forms for many projects. Second, the weight of the cap elements are reduced due to the weight of the concrete that was used to define the variable beam seat elevations and the area between them. Additionally, the risk of dimensional (elevation) error of beam seats during erecting was greatly reduced by the ability to easily adjust the beam seat bearing block elevations in the field. In 2014 we employed this generation of details successfully on the IH 39 corridor.

In addition to the simplification of the precast pier details, we also made the decision to have the cast in place and precast configuration and details be identical in geometry and details. In doing this, were

recognized the opportunity to promote consistency in our standards and provide for the opportunity for interchangeability. The benefit of this approach would be that if a decision is made to change from cast-in-place to precast construction (or, vice versa), there would be no need to redesign the configuration of the structure to make this accommodation. This capability will provide project staff and contractors utility and options in implementing projects and mitigating project risks related to pier construction.

INSTITUTIONALIZATION

In the development of the progressive generations of precast pier elements, we drew upon all of the stakeholder's knowledge and experiences in the design, fabrication, and construction process. As we developed corridor and general application standards, the ability to implement them in a way that met project needs throughout the life cycle of the project (planning, design, & construction) was important. Given that the precast standards were developed in a manner that now reflected a cast in place versions that we use on any application for improvement projects, it made a lot of sense that the precast and cast in place versions would be interchangeable. We added a note to the standard to be used on plans that allowed the substitution of precast piers in lieu of cast in place by the contractor. Our thought was that the department may mandate their used based on our understanding and need to mitigate risk for a given project, however, the contractor may elect to use this technology to help control their schedule, minimized their traffic control costs, or ensure their workers safety.

In developing the precast and cast in place standards to reflect common details and geometry, we have allowed the ability to introduce or interchange one for the other at any point during project design or construction. One benefit of the interchangeability is that there is no need to reconfigure the other bridge details to accommodate the change. Also, in interchanging it allows both the department and the contractor to benefit from this feature. The department befits from the ability to control risks to operations and safety while the contractor benefits from the ability to control their schedule.

We have also provided a plan note that allows the precast substitution to be paid for at the cost of the cast in place option thereby eliminating contract negotiation during construction.

THE CONTRACTOR MAY FURNISH A PRECAST PIER (CAP AND COLUMNS ELEMENTS) IN LIEU OF THE CAST-IN-PLACE PIER WITH THE ACCEPTANCE OF THE SHOP DRAWINGS BY THE STRUCTURES DESIGN SECTION.

THE PRECAST PIER SHALL CONFORM TO PRECAST DETAILS IN CHAPTER 7 STANDARDS OF THE CURRENT WISCONSIN DOT BRIDGE MANUAL.

PAYMENT FOR THE PRECAST PIER SHALL BE BASED ON THE QUANTITIES AND PRICES BID FOR THE ITEMS LISTED IN THE "TOTAL ESTIMATED QUANTITIES".

General Notes placed on Pier Plan Sheets

LESSONS LEARNED

The lessons learned though the development of our precast piers are reflected in this abstract and the accompanying presentation for the 2015 National Accelerated Bridge Construction Conference. Most all of the lessons were constructive lessons that were the benefit of the close relationship and coordination with the various project stakeholders that included researchers, designers, fabricators, contractors, and the Federal Highway Administration. Through this development and implementation process, we were able to address project needs and promote safety and lesson the disruptions to operations.

G-2: ABC PROJECTS WITH GRS-IBC

FIRST INNOVATIVE APPLICATION OF GRS-IBS IN THE GARDEN STATE

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INTRODUCTION

Gloucester County Bridge #4-H-5 carries Jessup Mill Road over Edward's Run, in Mantua Township, New Jersey. Built in 1925, the existing 20 foot-long reinforced concrete arch culvert was structurally deficient and was closed to traffic in August 2013, forcing a 6-mile detour. With many stakeholders impacted by the closure, the County sought an immediate solution that would accelerate reopening the vital link.

Pennoni provided design of the replacement bridge utilizing an unconventional concept never before used in the State of New Jersey: Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS). The GRS-IBS system was selected to minimize construction cost and duration. This project was constructed between December 2014 and March 2015 and was open to the public on April 7, 2015. Pennoni designed the new bridge with a prefabricated superstructure consisting of adjacent prestressed concrete slab beams and a composite reinforced concrete deck supported on GRS-IBS substructure. The GRS-IBS is part of a nationwide campaign for Accelerated Bridge Construction and Every Day Counts Campaigns and highly suggested by FHWA.

The new bridge was constructed within the footprint of the old bridge to maintain both vertical and horizontal alignment. The site is bounded on the north by the wooded flood plain of Edwards Run and a residential property, on the east by densely wooded area, on the south by the flood plain of Edwards Run and NJDEP designated wetlands, and on the west by residential properties. The Preliminary Bridge Type Study evaluated Prefabricated Reinforced Concrete Rigid Frame (Three Sided Culvert) and Prestressed Concrete Adjacent Box Beams supported on GRS-IBS system. The new span is 22 feet long and 32 feet wide to accommodate two 11 foot lanes and 5 foot shoulders. The U-Shaped GRS abutments and wingwalls kept the new bridge within the limits of right-of-way. FHWA approved side mounted guiderails were utilized to maximize the bridge width. Jessup Mill Road is approximately 13.5 feet above the Edward's Run channel bed. The 17 inch thick prestressed concrete superstructure provided the greatest hydraulic opening of the alternatives bridge types evaluated. Sheet piles were installed in front of

the abutment for excavation support and water diversion during construction. After construction, the sheeting was cut at the mud line as an added scour countermeasure in combination with riprap.

Both superstructure and substructure were constructed with light weight construction equipment to shorten construction duration, resulting in further indirect cost savings to the project and public.

FOUNDATION

The GRS-IBS alternative requires a low allowable soil pressure at the bottom of the footing, on the order of 3 ksf, without a need for a deep foundation. This is based on the assumption that unsuitable soil will be replaced with unreinforced concrete or select fill. Permanent sheeting was used as cofferdam to minimize channel excavation and for dewatering of foundation excavations. The sheeting was cut one (1) foot below the proposed channel bed to provide additional protection against potential future foundation undermining due to scour.

Based on the results of our field exploration and laboratory testing, we concluded that portions of the existing on-site soils were loose to a depth of approximately 23 feet below the existing roadway surface and not suitable for support of the conventional structure on shallow isolated and/or continuous spread footings in their existing conditions. The proposed GRS-IBS bridge bottom of footing was founded at approximately 15 feet below grade, leaving approximately 7.5 feet thick loose soils underneath the proposed foundation. Initially it was proposed that the loose soils can be addressed by implementing a soil exchange or ground improvement such as compaction grouting. Since the GRS-IBS system requires minimum allowable bearing pressure, the underlying soil layer did not need to be improved.

Additionally, foundation settlement of up to one inch was considered acceptable for the GRS-IBS bridge alternative. Total settlement for the soil profile below the bottom of the proposed bridge was estimated to be less than 1 inch. Due to the soil matrix, settlement for the cohesionless soils occurred relatively rapid and concurrently with load application and post-construction settlements was negligible.

GRS-IBS

Grade differential and soil conditions were the main considerations to use GRS-IBS abutments. The GRS-IBS system utilized for this project was comprised of four typical components, starting from the bottom: mud slab in lieu of typically used Reinforced Soil Foundation (RSF), General GRS Abutment, Bearing Zone, and Integrated Approach Zone.

The RSF is typically aggregate reinforced with intermediate geotextile and then encapsulated geotextile – for this project a mud slab was utilized to prevent water infiltration from below. This acts like the footing of the abutment. The General GRS Abutment is made of CMUs, geosynthetic reinforcement, and

reinforced soil. Blocks were leveled and soil was placed and compacted. A pulled-tight geosynthetic fabric was then placed on top of the compacted soil overlapping half of the block. This process was continued until the Bearing zone. Reinforcement geo synthetic fabric was spaced at 8 inches. Soil was compacted in four inch lifts to minimize the damage to the geosynthetic. The first three feet closest to the wall was hand-compacted. Since the abutments are spanning water, solid blocks of CMU were used up to high water level. Temporary cofferdam sheeting was cut below the channel bed and riprap was placed in front of the CMU wall to further protect the abutment against scour.

The top five layers of geosynthetic reinforcement are considered the Bearing Zone. The Bearing Zone is similar to the General GRS Abutment, except there is an intermediate layer of geosynthetic fabric. Also, the top three rows and all rows on the corners are filled with concrete and rebar reinforcement. The top CMU received a 1½ inch concrete cap.

The Integrated Approach Zone is located from the bottom of the beam to the top of grade. This zone does not have any CMUs, instead the primary reinforcement was wrapped around the compacted soil behind the stub abutments. There were two compaction layers with secondary reinforcement between layers of primary reinforcement. The primary reinforcement extended four feet beyond the secondary reinforcement. All geosynthetic reinforcements in this zone extended three feet beyond the 1:1 reinforcement soil slope. There was a two inch clear space between the last layer of reinforcement and the asphalt.

CONSTRUCTION COST

Our estimated construction cost for this project was $600,000. There were six contractors that bid on this project. Their bids ranged from $840,000 to $1.5 million. All the contractors unbalanced their bids by bidding high on bridge demolition. This allowed them to receive front loaded compensation. The bid price for the GRS-IBS items varied from $7 per square feet to $165 per square feet. The lowest bidder, at $840,000, was selected for the project. His bridge removal price was ten times higher than the engineer’s estimate, superstructure twice as engineer’s estimate but the GRS-IBS abutment and u-wings price of only 13% of the engineer’s estimate. The total construction cost of the project was approximately $900,000 with claims and extras such as relocation of water main, rearrangement of cofferdam sheeting due to wider than expected abutment footing, and construction of structural guiderail and its moment slab base.

SCHEDULE

The contractor's original schedule was based on a January and February construction. The GRS-IBS abutments and u-wings were scheduled to take two and half weeks. The actual construction extended into March and the GRS-IBS abutments and u-wings were constructed in four weeks. The construction delay for the most part was caused by the extremely cold and wet winter. One other factor that slowed to contractor down was the integration of the corrugated polyethylene tubes for the installation of the deep guiderail post immediately outside four corners of the bridge superstructure.

LESSON LEARNED

The lay down area behind the guiderails along the u-wings was less than the recommended four feet. This necessitated the addition of structure mounted guiderail along the u-wings and shifting the guiderail to barrier connection segment outside of the u-wings limits. The structural mounted guiderails had to be supported by reinforced concrete moment slabs.

CMU blocks were specified with required a compressive strength of 4,000 psi. This requirement, although not necessary, made it harder for the contractor to use CMU blocks produced by local supplier. Actual CMU block sizes are 7 5/8", allowing for 3/8" mortar joint, which adds up to 8". The dry staked CMU walls needed an extra layer of CMU.

CONCLUSION

GRS-IBS abutments and u-wings were chosen for this foundation system to reduce the impact to wetland, right-of-way, and embankment excavation. This system minimizes foundation pressure by spreading the superstructure load through the soil embankment.

However, for this project, the cost-savings and short construction schedule were not optimized as planned. The contractors' unfamiliarity with the GRS-IBS system caused all six contractors to bid high. The anticipated time savings on such a small project was approximately 30 days and that was hindered more by the harsh 2015 winter climate than anything else. There were a couple of other challenges under current project procurement methods that restricted cost-effectiveness of this system. The GRS-IBS system requires small equipment and low-skilled labor. Normally, the type of contractors that pursue bridge projects are mid-size transportation construction companies that have access to large and small equipment, their labor force is highly skilled, and are carrying large insurance premiums. Instead, the cost savings for such projects lies in utilizing bridge owner's small maintenance force. In time, as more of

such projects are constructed, contractors' will become familiar with GRS-IBS as bridge construction method, and the goal for cost saving and time savings will be met.

GEOSYNTHETIC REINFORCED SOIL-INTEGRATED BRIDGE SYSTEM (GRS-IBS) IMPLEMENTATION IN MISSOURI – LESSONS LEARNED

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ABSTRACT

This report explains the basics of an accelerated bridge construction method which has very recently been used for the first time in Missouri and not yet in widespread use across the country. This construction method results in a bridge having an integrated super and substructure in what is called a Geosynthetic Reinforced Soil Integrated Bridge System or GRS-IBS.

The presentation will delve further into the explanation of the GRS-IBS by telling of the experiences had using this accelerated bridge construction technique in the State of Missouri. One project in particular, Rustic Road over the North Fork Grindstone Creek in Columbia, Missouri exemplifies the challenges, intricacies, and eventual successes to be had when utilizing this type of bridge abutment.

NOTATION

CMU	Concrete Masonry Unit
GRS-IBS	Geosynthetic Reinforced Soil Integrated Bridge System
FHWA	Federal Highway Administration
IBRD	Innovative Bridge Research and Deployment
MoDOT	Missouri Department of Transportation

INTRODUCTION

The Geosynthetic Reinforced Soil Integrated Bridge System or GRS-IBS, is an innovative construction technique currently being promoted by the Federal Highway Administration to assist agencies who are looking to save construction costs and time to construct. The thought is that since the construction methods are relatively simple and do not require specialized machinery, agencies can build these

structures with their own forces, Adams, et al. (1). FHWA provides an implementation guide for use in design and construction techniques.

Another benefit to the GRS-IBS is it can reduce construction time as compared to conventional methods. The GRS-IBS abutment requires no concrete and can be constructed in weeks rather than months, Adams, et al (1). Once constructed, the GRS abutment is ready to set girders on top immediately without waiting for curing time associated with conventional concrete abutments.

The presentation will discuss the design challenges of the new methods and products, including the unique details developed and how the construction process could be further improved. Additional lessons learned on successfully bidding this innovative technology will be discussed to assist future implementation in managing budget concerns. Finally, post-construction testing will be presented showing how the abutment has been performing and analyzing the longevity of the system.

GRS-IBS BRIDGES IN MISSOURI

To date, three bridges have been built in Missouri utilizing the GRS-IBS construction methods. One bridge, constructed by MoDOT is a grade crossing located on Route B over Business Loop 70 in Columbia, Missouri. Randolph County constructed a GRS-IBS structure over a railroad. The third bridge constructed was built on Rustic Road in Boone County Missouri and was a joint venture between the City of Columbia and Boone County. Bartlett & West assisted the County and City in making this project a reality by developing an inventive construction option and helping to secure Innovative Bridge Research and Deployment (IBRD) funds.

THE RUSTIC ROAD BRIDGE

The Rustic Road bridge crosses over the North Fork of Grindstone Creek on the border of the city limits of Columbia in Boone County, Missouri. The existing bridge was a highly deteriorated steel bridge on a dead end road. The load posting across the bridge had been reduced to a point that school buses and fire trucks would no longer cross the bridge which made it difficult and potentially dangerous for the residents on the far end of the road.

The main goals of the project were to replace the deteriorated structure and to do so with the least amount of inconvenience to the residents. Therefore the time that the road was closed had to be as short as possible. The best way to shave time off of bridge construction is to eliminate on-site concrete pours. This can be done either by using precast concrete elements or by utilizing methods that do not require structural concrete.

To demonstrate the speed that a small bridge could be constructed and to facilitate the requirements of the residents of Rustic Road, the IBRD grant required the use of the GRS-IBS abutments as well as prefabricated steel tub girders which had not yet been widely used in the State of Missouri. The CONSTRUCT™ girder system was used for the single span, which again eliminated the need for cast-in-place concrete. This superstructure system uses steel tub girders with a concrete slab poured at a precast facility. The units were then shipped on site and set on the GRS abutments. The girders do not require an overlay and therefore can be driven on as soon as they have been placed and secured. Additional details were developed to accommodate the intricacies of this system, such as handling any floatation concerns.

The University of Missouri was another partner on the project. Bartlett & West worked with the university to include the installation of testing devices in the project's design. The devices allow continual monitoring of the abutments, including the lateral and vertical movement of the abutment system.

This project served as one of the pilot projects for the system in Missouri. It was used to promote the GRS technology and as a test for potential future applications of the GRS-IBS construction techniques.

GRS-IBS COMPONENTS AND DESIGN –LESSONS LEARNED

A Geosynthetic Reinforced Soil substructure is made up of alternating layers of aggregate and geotextile reinforcement which act as the abutments of the bridge. These alternating layers of aggregate and reinforcement are built up until the abutment is the proper height to place the girders on. This system is not supported externally and therefore the facing elements are not considered a structural element in the design, Adams, et al. (1).

MAIN REINFORCEMENT ZONE

The main reinforcement area is where the aggregate material is layered with a high strength geosynthetic material. FHWA recommends an ultimate strength of 4,800 lb/ft for GRS load bearing capacity. Serviceability is assured by limiting the required reinforcement strength to less than that of the geosynthetic at 2% strain, Adams, et al. (1). These values are readily provided by geosynthetic material manufacturers.

The specifications for Rustic Road allowed for flexibility in geosynthetic material choice. Specifications were drafted such that either a geotextile or geogrid material could be chosen. Also, the material could have the required strength in two axis or one so long as the material was properly placed. The hope was that by giving flexibility in the material choice, pricing for the geosynthetic material could be kept low.

FACING BLOCKS

The facing blocks, even though they are not structural elements, became an issue for all the Missouri GRS-IBS projects. The concern was freeze-thaw cycle testing and the overall longevity of the façade.

One of the core principles behind the GRS-IBS construction method is to allow agencies to build these abutments with their own forces. To allow for this, the smaller CMU blocks are the preferred facing unit since they can be placed by hand. MoDOT has specifications that require substantial testing and verification when using the dry cast CMU style of block due to past projects that deteriorated from a combination of freeze-thaw issues as well as salt spray. Since Rustic Road does not get salt applications due to the low traffic, the salt test was removed from the specification requirements. According to research conducted by Bartlett & West, this shaved at least 1-2 months off of the freeze-thaw testing time.

The other two GRS bridge projects opted to use a larger wet cast facing block. These type of blocks can weigh upwards of 400 pounds apiece and therefore cannot be placed by hand.

Superstructure

The superstructure consists of steel tub girders which are manufactured under the CONSTRUCT™ name. These girders were an integral part of the IBRD grant because they allowed for quick construction and would therefore lessen the impact to the residents of Rustic Road.

The particular configuration of these girders, having a hollow steel tub, raised concerns with potential floatation of the superstructure during frequent flood events. The normal solution is to simply use anchor bolts and tie the girders to a concrete beam cap. However, with the GRS abutment there is no beam cap to anchor to.

This challenge led to an innovative solution of using anchor bolts secured to steel plates that were then buried into the aggregate of the GRS abutment.

Integrated Approach

The Integrated Bridge (IBS) portion of this construction method is unique in that it is designed to eliminate the “bump at the end of the bridge”. Since the roadway and the bridge superstructure are both supported on the GRS material, there is no differential settlement which is common for conventional bridge abutments on piles where the roadway settles and the abutment does not.

The FHWA Guide calls for a layer of asphalt to be put down as a wearing surface on the bridge and to tie into the roadway asphalt. For the Rustic Road application, due to the fact that the supplied girders do not

require a wearing surface, and to save construction time, this asphalt was not placed which did leave a joint between the girder backwall and the integration zone. Care was taken in the specifications to have a geosynthetic with adequate drainage properties to avoid excess water retainage in the abutment.

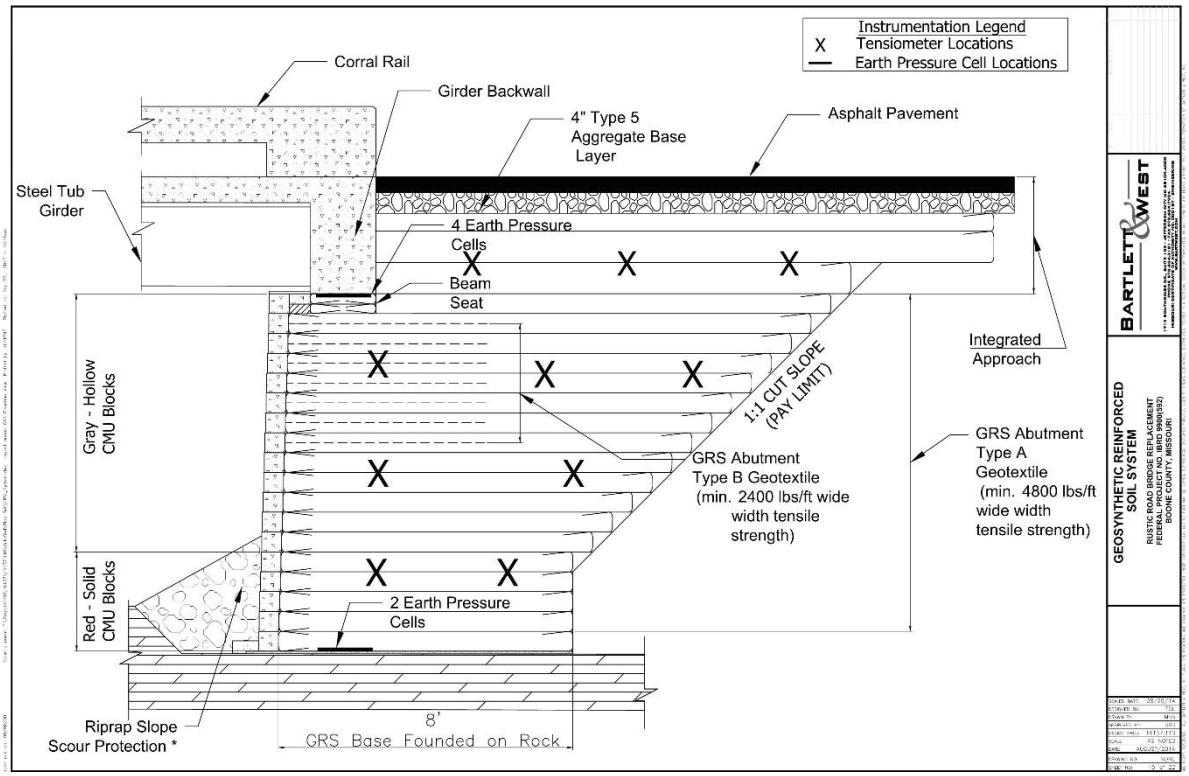

Figure 1: Rustic Road GRS-IBS abutment

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PHASED ACCELERATED BRIDGE CONSTRUCTION WITH GRS/IBS ABUTMENTS

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EXTENDED ABSTRACT

This presentation describes the design and construction of a bridge replacement project that included phased bridge construction and the use of Geosynthetic Reinforced Soil Integrated Bridge System (GRS/IBS) for the abutments. Typical design details for phased construction of a bridge using the GRS/IBS abutments are not included in the FDOT's Developmental Design Standard Index D6025 for GRS/IBS abutments nor the FHWA publication - Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide (Pub. No. FHWA-HRT-11-026). This presentation provides some lessons learned regarding the design and construction of a GRS/IBS abutment on a phased accelerated bridge replacement project.

PROJECT DESCRIPTION

This project involved the replacement of the Rock Ridge Road Bridge over the Gator Canal in Polk County, Florida under a design-build contract with the Florida Department of Transportation (FDOT). Rock Ridge Road is a rural road just north of Lakeland in Polk County, Florida. The existing structure was a 2 span, 37 foot long bridge spanning Gator Canal. The curb to curb width was 29'-7" and prior to construction the bridge was posted for 16 tons for a single unit truck. The existing bridge skew did not coincide with the roadway alignment. It appeared the original bridge was designed with a 90 degree angle of crossing and a subsequent roadway improvement project introduced a curve on the bridge. The superstructure consisted of precast, prestressed, channel beams with cast-in-place concrete filler between the beams with a concrete deck. The substructure bents consisted of prestressed concrete piles and cast-in-place concrete caps. The roadway alignment across the bridge was on a 7 degree curve with a 747 foot radius.

PROJECT REQUIREMENTS

The FDOT design-build Request for Proposal (RFP) included the following requirements:

To provide 2 - 12 foot lanes and 8 foot shoulders

To maintain the existing roadway alignment and profile

Included minimal roadway reconstruction – stipulated a minimum of 50 feet of rehabilitation

Required traffic barriers that provided a minimum of a TL-4 standard

Maintenance of Traffic per FDOT Index 606 (temporary traffic signals and alternating one-way traffic)

The schedule allowance was 470 calendar days for design and construction

ACCELERATED BRIDGE CONSTRUCTION (ABC) APPROACH

Due to tight right-of-way constraints and an unacceptably long off site detour, the new bridge needed to be designed in phases using a one lane, two way alternating traffic pattern with temporary traffic signals to maintain traffic. The high cost and risk associated with this type of traffic control system convinced the design-build team, led by Wright Construction Group, Inc., to incorporate ABC techniques into the project approach to reduce the cost and duration of construction. Wright Construction's bid was 19% less than the 2nd lowest bidder and 40% less than the highest bidder.

A key element in our ABC approach was to minimize the use of cast-in-place (CIP) concrete bridge components. The only CIP concrete elements incorporated in the design of this bridge were the F-shaped concrete barriers and the 2'-9" wide closure pour for the prestressed slab units between phases. The design build team's ABC design included the following primary components.

Prefabricated Concrete Superstructure

The superstructure consisted of precast, prestressed concrete adjacent slab units. Each of the slab units were detailed to be the same size at 18" x 3'-8" x 45'-7". To further speed up construction, we eliminated the FDOT standard 6" CIP composite concrete deck that is part of the FDOT Developmental Design Standard Indexes D20350 & D20360 Series for Prestressed Slab Units. We designed a transverse post tensioning system in lieu of the concrete deck to provide for the lateral live load distribution. The transverse post tensioning system was designed using high strength steel rods in accordance with AASHTO 5.14.4.3 and the FDOT Structures Design Guidelines 4.4. The design plans specified a 2" thick asphalt overlay over a membrane waterproofing system to protect the beams from water intrusion. The design of the post tensioning system included providing a 250 psi pre-compression on the grouted shear key joints. We used 1.25" diameter Dywidag Bars at a 7' spacing with each bar tensioned to 131 kips.

GRS/IBS Abutments and Wingwalls

The GRS/IBS abutments and wingwalls reduced the project cost and construction duration compared to a conventional pile bent abutment in accordance with FDOT standards which was proposed by the other three (3) design build teams. The use of GRS-IBS abutments eliminated deep pile foundations, bearings, expansion joints, CIP concrete backwalls, CIP concrete caps, and CIP concrete approach slabs that are included in the FDOT standards.

We incorporated design details from FDOT's Developmental Design Standard Index D6025 for GRS/IBS abutments to reduce the design cost and time. The woven geotextile reinforcement options are listed in FDOT Design Standards Index No. 501 and the requirements were provided in the plans. The product we selected was Mirafi® HP570 which is composed of high-tenacity polypropylene yarns. The gravel can be graded aggregate base material (GAB) meeting the requirements of Section 204 and/or coarse aggregate comprised of natural stones meeting the requirements of Section 901 with a size distribution of any of the listed aggregate gradations from Size No 57 through Size No 89. We decided to use #57 stone.

The FDOT Developmental Design Standard Index D6025 specify Concrete Masonry Units (CMUs) that have 8"x 8" x 16" nominal dimensions. The design build team alternatively selected to use the Vertica block manufactured by Oldcastle Coastal which is a licensee of Anchor Wall. The Vertica block has a built in 2 degree batter and the nominal dimensions are 8" x 11" x 18". The interface connection for the CMU's rely on friction while Vertica blocks have a built in male-female key to enhance the connection. Standard CMU blocks are not manufactured to same dimensional tolerances as the Vertica block. Vertica blocks have tighter tolerances because they are always laid up dry where standard CMU blocks are typically laid up using mortar. The Vertica block provided enhanced aesthetics compared to the CMU blocks and the standard concrete mix for Vertica is 4,000 psi. The Vertica block is also less prone to impact damage at the front face due to its thicker front wall face.

PHASED CONSTRUCTION OF GRS-IBS ABUTMENTS

Typical design details for the phased construction of a bridge using the GRS/IBS abutments are not included in the FDOT's Developmental Design Standard Index D6025 for GRS/IBS abutments nor the FHWA publication - Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide (Pub. No. FHWA-HRT-11-026). Therefore, we needed to develop design and construction details to allow for the construction of the GRS-IBS abutments in 2 phases.

This project required the use of temporary steel sheet piling (XZ-95) to support the roadway embankment during the excavations for the GRS-IBS abutment during Phase 1 and Phase 2 due to the very limited

roadway right of way compared to the proposed out to out width for new bridge. In addition, the GRS/IBS abutment needed to be constructed immediately adjacent to the sheet piling to maintain the minimum required detour roadway width on the portion of the original bridge that we needed to maintain traffic while we constructed the first phase of the new bridge. The sheet piling was designed for a retained height over 10 feet which required an embedment depth of 22’-9” below the bottom of the excavation.

We considered leaving the steel sheeting in place but dismissed this approach for reasons that included:

This approach would have created a discontinuity between the two phases of the GRS abutments.

We were concerned about differential settlement of the slab units near the interface with the buried steel sheeting and the GRS abutments.

We were concerned about the development of cracking in the asphalt pavement above the interface with the buried steel sheeting and the GRS abutments.

The spaces formed by the Z shaped sheet piling needed to be filled in using concrete or steel sheets to provide a flush face to contain the edge of the GRS abutment along the sheeting.

The additional cost of the sheeting and installation of the filler material.

The preferred approach was to be able to remove the steel sheeting after the completion of the GRS abutment in Phase 1 to allow for continuity with the construction of the GRS abutment in Phase 2. The challenges to this design approach included:

Supporting the new roadway embankment from Phase 1 to allow for the excavation and the construction of the Phase 2 GRS abutment.

Removal of the steel sheeting without causing damage to the new Phase 1 abutment elements.

To support the roadway embankment during the Phase 2 construction, a temporary GRS wall was constructed along the face of the temporary sheeting during Phase 1. The temporary GRS wall was built by overlapping the geotextile sheets 3 feet back from the face of the steel sheeting. See Figures 1 and 2.

Figure 1 - Phase 1 Sheeting

Figure 2 - Phase 2 GRS Temporary Wall

Sheets of 1” thick plywood were specified to be placed temporarily between the steel sheeting and the temporary GRS wall to allow for the removal of the sheeting without damaging the temporary GRS wall face. In addition, the last 3 sections of temporary sheeting were cut off at the bottom of the excavation to minimize vibrations during extraction next to the new block facing units in the new GRS/IBS abutment for Phase 1. The temporary plywood also provided a flush face for the edge of the GRS abutment instead of the varying profile of the Z shaped steel sheeting. It also provided some insulation for the geotextile of the temporary GRS wall from the heat caused by flame cutting the last 3 sections of sheeting at the bottom of the excavation. The details from the contract plans are shown below i Figures 3 and 4. GRS/IBS abutments are constructed using a running bond (or staggering of the vertical joints) in the facing blocks. To facilitate the construction joint between phases in the abutment, we specified a butt joint at this joint.

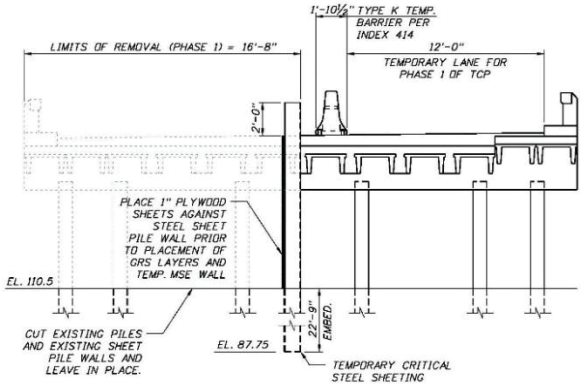

Figure 3 – Phase 1 Details

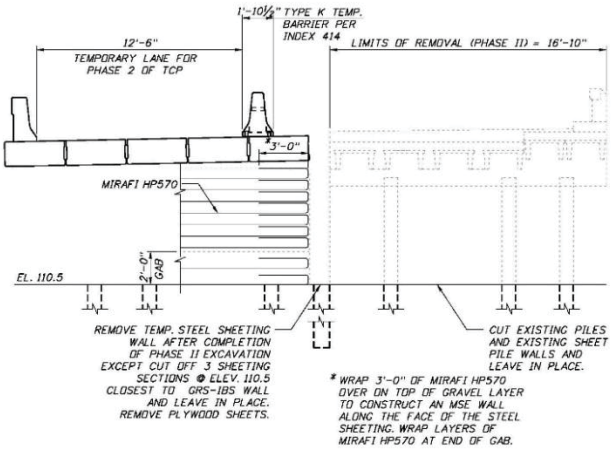

Figure 4 – Phase 2 Details

G-3: EVALUATION AND MONITORING OF ABC BRIDGE MOVES

STRUCTURAL RESPONSE MONITORING TO ASSESS BRIDGE CONDITIONS DURING CONSTRUCTION

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ABSTRACT

Structural Response Monitoring combines the advanced capabilities of vibration monitoring with the simplicity of static monitoring for measuring structural responses. Any change in structural response indicates either an altered loading environment or a change in a structure's ability to manage loading. A Structural Response Monitoring system provides concise data outputs that describe the current state of the structure and how structural elements are responding to working and environmental loads. A Structural Response Monitoring system was installed on the Highway 61 Hastings Bridge that spans the Mississippi River in Hastings, Minnesota during the construction of a new, adjacent bridge. The objective of the installation was to monitor the responses of the existing bridge during the construction period, and verify that the existing bridge was not being negatively affected. Impact, tilt and vibration responses were calibrated against loading and environmental conditions to create boundary conditions for continuously assessing measured performance against expected performance. In particular, substructure responses caused by changes to loading and support conditions were measured to a level far beyond the capability of traditional monitoring and survey equipment. When the river rose in the spring, the changes in pier responses were easily identified and when the water receded, the pier responses returned to their former values. The Structural Response Monitoring system allowed the construction team to better understand the everyday in-place performance of the existing bridge. The system's performance capabilities helped the owner establish a very high level of confidence for assessing onsite conditions in near real-time. The monitoring system reduced the amount of field surveying necessary and acted as a key indicator for potential movements and subsequent supplement survey demands. Due to the inherent assembly process utilized during Accelerated Bridge Construction (ABC), as well as the emphasis on understanding and certifying substructure performance and capabilities, the installation of a Structural Response Monitoring system during ABC, provides useful data with regards to location of prefabricated members, tolerances for slide-in members, and settlement levels of foundational elements. The data helps avoid schedule and budget impacts caused by unnoticed movements or mismatching between prefabricated bridge elements

and the substructure. The system provides the contractor and owner near real-time information for making dynamic data driven decisions.

PROJECT OVERVIEW

This project was a Design/Build replacement for a 60 year old structurally deficient arch truss bridge over the Mississippi River in Hastings, Minnesota. The new 1938 foot long bridge would be located next to the existing bridge. This project presented many challenges for the team including:

The location for the new bridge was very close to the existing bridge.

Construction activities could only cause minimal disruptions for the 30,000 cars that used the bridge daily.

Foundation prep work for the new bridge, had the potential to disturb the existing bridge's substructure.

The current bridges configuration and location caused it to be a navigation hazard and increased its risk for vessel collision.

Frequent historical flooding of the Mississippi River.

Piers in the main channel were scour susceptible.

A land-based pier near the construction staging area was prone to subsidence.

In order to help the team identify undesirable events and behaviors, a Structural Response Monitoring system was specified to be installed on the existing bridge.

STRUCTURAL RESPONSE MONITORING

Structural Response Monitoring (SRM) combines the advanced capabilities of vibration monitoring with the simplicity of static monitoring into one system that measures how a structure reacts to loads and support conditions. The system monitors how structural elements, vibrate, deflect, tilt, twist and settle under working loads as well as environmental loads. The operating premise for the system is that in order for a new response to occur, it has to be driven by either a change in the loading that is being applied to the structure, and/or a change in the support capacity of the structure to cope with loading. The instrumentation that is used for SRM centers around the use of a specialized sensor module that contains 3 accelerometers, 2 tilt meters, and 1 temperature sensor.

SRM CONFIGURATION FOR THIS PROJECT

The risk analysis for this project indicated that due to the considerable amount of foundation work required for the new bridge and its close proximity to the existing bridge, the substructure elements of the existing bridge were especially vulnerable to potential construction influences and the volatility of the Mississippi river. 4 piers in particular were identified as being especially vulnerable; previous inspections indicated that 2 piers were scour susceptible, and 1 pier was prone to subsidence. An additional pier was identified for monitoring due to its very close proximity to an area that required significant substructure support activity. The final system configuration for this project consisted of: 4 sensor modules, 1 data collection appliance, 1 data extender module, 5 cables, and because the electrical service on the bridge was not reliable, an uninterruptable power supply (UPS) was used to power the system. The system was installed in less than 4 hours and provided continuous service for the duration of the 30 month project.

SRM ANALYSIS ROUTINES

Once the system was installed, it was configured to continuously sample all of the sensors at the rate of 400 samples per second. Once received; sampled data was then processed for publishing. When an undesirable response was detected, a notification was automatically dispatched. The typical processing routines included:

Baseline vibrations and vector responses

Vibration and vector responses were indexed for the various environmental and traffic conditions. A predictive response routine was employed to identify deviations.

Pier tilt vs water flow and rate

Tilt angles were indexed to local hydrological readings. In-water piers demonstrated a consistent response relationship when water level and flows were elevated. This relationship and the elastic response of the piers was used to index expected and unexpected behaviors.

Pier tilt vs. temperature

Tilt angles for all piers, displayed a strong dependency to temperature. This behavior was mapped and used to identify variations related to loading and support.

Impact responses

Vector and frequency analysis routines were used to identify: when impacts to the structure occurred and the damage potential a specific impact could impart.

SRM DATA APPLICATIONS

As noted earlier the SRM system is a continuous recording system and does not require a user to define parameters to trigger recording. This mode of operation produces a continuous data set that can be viewed in near real-time and provide a quantifiable index for evaluating how the bridge is reacting to current circumstances. Some notable events that occurred during the project where the SRM data was used to help qualify their influences included: statnamic testing, pile driving, construction accident, flooding, extreme wind, frozen bearings, and subsidence.

SRM ADVANTAGES

For this project an SRM system proved to be a valuable asset for the team. In addition to the system's ability to deliver understandable and actionable data, some other realized value points were:

Significantly lower system acquisition and operating costs

Reduces on-site survey demands

Very fast installation with little site prep required

Reusable system that can easily be configured for the next project

High resolution data that identifies settlement and subsidence conditions well before other detection systems

SRM FOR ABC

The realized value that the SRM system delivered for this project demonstrates a performance capability that can provide a useful advantage for certain aspects of ABC. Within ABC, substructure and deck support condition assessment is often an important aspect of a project. An SRM is an easy-to-use solution that can provide an extra measure of awareness and control for the following scenarios:

Verifying that adjacent construction activities are not negatively impacting nearby structures.

Confirming the loading and response behavior of existing substructure elements that are going to be reused.

Measuring the loading and settlement responses when setting or marrying major bridge assemblies.

Confirming deck support and substructure performance operates and remains within the expected ranges.

Confirm the handling of major assembly and qualify responses related to unintentional events.

Measure critical alignments and record settling responses.

An SRM system can provide some distinct advantages for ABC projects by providing an easy-to-use system that can continuously monitor and compare how structural elements are responding to their environment. The system can detect the early responses that are indicative of poor performance and help minimize their impacts to budgets and schedules.

FRICTION VALUES FOR SLIDE-IN BRIDGE CONSTRUCTION

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INTRODUCTION

During Slide-In-Bridge Construction (SIBC), bearing pads consisting of polytetrafluoroethylene (PTFE), commonly known as Teflon, and stainless steel surfaces are often used during the sliding process. Appropriate Coefficients of friction (CoF) had not been determined for the use of SIBC. Additionally, it was unknown how the CoF would be affected as parameters were varied such as the roughness of the stainless steel surface, lubricants used on the contact surface, pressure applied, and sliding speed of the bridge.

LABORATORY TESTING

Researchers at Utah State University carried out multiple tests to determine the effects of these parameters on the CoF during bridge slides. Tests were performed using Type 304 Stainless Steel with two different levels of surface roughness ranging from rough to smooth. Tests were also conducted using Carbon Steel in order to determine how well the bridge would slide if the girders were placed directly on the PTFE pads. This option would eliminate the need for contractors to purchase and install stainless steel sheets for the sliding surface. Various lubricants were investigated: silicone grease, motor oil, graphite, dish soap and a dry surface for baseline comparison. Additionally, four different applied pressures and two different speeds were investigated. With this test matrix there are 120 combinations that could be explored. To narrow down the amount of tests needed to obtain valuable results investigative decisions were made based on results obtained to determine if a particular variable would be tested at more length. Narrowing down the number of tests this way produced around 40 combinations in which performance data were collected and used for comparison. The test matrix is shown below in Table 1.

TEST PARAMETERS INVESTIGATED

The decision to investigate the four parameters outlined was based on past research others have done, current AASHTO specifications and current construction practices.

Roughness of Stainless Steel

Type 304 stainless steel was used for the testing with two different surface finishes: #2B finish and #1 finish. The stainless steel with a #2B finish is the most common finish available and is usually less expensive than other types of finishes. The performance for the #2B finish was similar to results obtained by Stanton and Taylor (1) for #8 mirror finish. A #8 mirror finish is the smoothest finish offered. Since SIBC standards and codes have not been fully developed many of the engineers and contractors using SIBC methods use #8 mirror finish based on the AASHTO specifications (1) for sliding bearings used as permanent bridge bearings. Stanton and Taylor (2) determined that #2B finish has similar performance to a #8 mirror finish when no lubrication is used and results obtained in this study confirms their conclusion that #2B is an adequate substitution for a #8 mirror finish. The stainless steel plate with a #1 finish did not perform as well as the #2B finish did. Stainless steel with this finish is not manufactured as sheet stainless steel, but is manufactured as plate. This is important to note because stainless steel plate is 3/16 inch thick or thicker, whereas sheet stainless steel is 7 gage (about 3/16 inch) and thinner. If a contractor wishes to use the #1 finish it may cost over three times more than if a #2B stainless steel sheet is used instead, due to the thickness difference. There was considerable difference in the performance of the #2B and #8 finishes when no lubrication was used, but when lubrication was used the difference in performance between the two finishes was reduced significantly. Along with the stainless steel surfaces investigated, carbon steel was also investigated to be used as a sliding surface. The CoF obtained when using carbon steel was much higher than all of the stainless sliding surfaces when no lubrication was used. When lubrication was used, the CoF dropped considerably but was still higher than the CoF obtained when using stainless steel.

Sliding Velocity

The two speeds at which the bearing pads were pushed back and forth were investigated at a velocity of 2.5 in/min and 10 in/min. The 2.5 in/min was chosen to be one of the velocities investigated because the current AASHTO specs (3) specify to use this speed and this was also one of the speeds used by Stanton and Taylor (2). The 10 in/min was chosen in order to investigate the response at a speed closer to what contractors would actually use. Varying this parameter showed that as the speed increased the CoF increased as well. Even though lower CoF values are obtained when using the slower speed of 2.5 in/min, it is recommended that contractors be given a range of sliding speeds that can be selected based on the field conditions. The difference in the CoF between the two speeds is relatively insignificant when comparing the advantage of quicker construction time when using a faster speed, versus the advantage of a lower CoF at slower speeds.

Lubrication between Interfaces

The sliding surfaces were tested with no lubricants as well as with lubricants. There were four different types of lubricants used: silicone grease, automotive motor oil, graphite and dish soap. All of the lubricants used produced lower CoF than the tests that were dry. Tests using the grease and motor oil performed the best and didn't have the issue of gumming up like the dish soap did when sitting for long periods of time. All of these tests were performed using dimpled PTFE, which allows small reservoirs of lubricant to be stored and keep the surface lubricated for a longer period of time. It is recommended that lubricant be used in bridge slides and that it be used with dimpled PTFE. If dish soap is used it should be applied within 4 hours of when the bridge will be pushed.

Contact Pressure

The pressure applied to the contact surface by the vertical ram, simulating the weight of the bridge, was varied between four different magnitudes of pressure. The pressures used for the tests were 1500 psi, 3000 psi, 4500 psi and 8000 psi. Most bridge slides currently use contact pressure under 4500 psi. The team wanted to explore how the CoF reacts to a wide range of contact pressures which is why such a high pressure of 8000 psi was chosen to be investigated. The results were similar to what has been observed by other researchers (2). As the contact pressure is increased, the CoF decreases until the contact pressure exceeds a pressure of about 3000 psi, where the CoF then stops to decrease and remains relatively constant.

Table 1 Test Matrix

Test Matrix		Lubricants Used between PTFE and Stainless Steel Surfaces				
Slide Speed	Pressure	Dry† (No Lubricant)	Silicone Grease (SAE-AS 8660)	Dish Soap	Automotive Motor Oil	Graphite
No.2B Rolled Finish						
2.5 in/min	1500 psi	X	X	X		
	3000 psi	X	X	X		
	4500 psi	X	X	X	X	X
	8000 psi	*	*	*		
10 in/min	1500 psi	X	X	X		
	3000 psi	X	X	X		
	4500 psi	X	X	X	X	X
	8000 psi	*	*	*		
Rough Rolled						
2.5 in/min	1500 psi	X	*	*		
	3000 psi					
	4500 psi	X	*	*	*	*
	8000 psi					
10 in/min	1500 psi	X	X	X		
	3000 psi					
	4500 psi	X	X	X	*	*
	8000 psi					
Carbon Steel						
2.5 in/min	1500 psi	*	*	*		
	3000 psi					
	4500 psi	*	*	*	*	*
	8000 psi					
10 in/min	1500 psi	*	X	X		
	3000 psi					
	4500 psi	*	X	X	*	*
	8000 psi					

X Primary tests to be performed
* Secondary tests will be based on results from primary tests
† Dry tests will be done with either dimpled or smooth PTFE

Primary tests to be performed:	34
Secondary tests to be performed:	26

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1. *American Association of State Highway and Transportation Officials (AASHTO). (2012). "AASHTO LRFD Bridge Design Specifications." Washington DC*
2. *Taylor, J.C., Stanton, J.F. (2010). "Friction Coefficients for Stainless Steel (PTFE) Teflon Bearings." Report SPR#0092-08-13. Wisconsin Department of Transportation, Madison WI American Association of State Highway and Transportation Officials (AASHTO). (2012). "AASHTO LRFD Bridge Construction Specifications." Washington DC*

SUPERSTRUCTURE MONITORING DURING SPMT MOVEMENT

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INTRODUCTION

Bridge numbers 01235 and 01236 carry East and West bound traffic on Interstate I-84 over Marion Ave near Southington Connecticut. The existing bridges were simple-span structures consisting of a concrete slab supported by 54 inch deep pre-stressed concrete girder. The Eastbound and Westbound spans were 102 and 103 feet respectively, both bridges were 56.5 feet wide. Originally built in 1963, each bridge carried three line of traffic with and average daily traffic greater than 83,000.

The superstructures, which were scheduled for replacement, had a condition rating of 4 (on a scale of 0 to 9) due to exposed reinforcement and tendons, longitudinal cracking beams, and spalled concrete at the end of girders. The substructures were able to be rehabilitated and reuse in conjunction with the replacement of the superstructure.

Contract specification required that Interstate I-84 only be closed for a weekend and proposed the use Self-Propelled Modular Transporters (SPMTs) to accommodate this timeframe. Use of SPMTs to install the new superstructures offered the ability to construct the new superstructures off site and install them in the required timeframe. The use of SPMT's also minimize traffic disruption, improve work-zone safety and lower overall construction cost when considering the reduction in construction time and disruption to traffic.

Northern Construction of Palmer, Massachusetts prefabricated the new superstructures on site in staging areas adjacent to the bridges. Traffic on Interstate 84 and Marion Ave was fully maintained during the entire prefabrication period using only temporary lane closures for shoulder and wingwall reconstruction.

The superstructure were replaced during the weekend of June 27-29 2014. Beginning at 5pm on Friday the 27th, Northern demolished the existing westbound superstructure, at the same time utilizing one of the SPMTs to lift and move the existing eastbound superstructure to temporary supports. Once the existing superstructures had been removed both SPMTs worked simultaneously to lift and move the new

superstructures to the existing foundations. Eastbound direction of Interstate 84 was fully re-opened to traffic at 4:40 p.m. and the westbound was re-opened at 8:15 p.m. on Sunday the 29th.

Although Barnhart, the SMPT subcontractor, monitored the bridge lifting and setting displacements via the SPMT's the span stresses and structural twists (torsion) needed to be monitored during movement to ensure over-stressing did not occur while minimizing the potential for cracking in the superstructures.

THE MONITORING SYSTEM

The instrumentation and monitoring program for the bridge move was designed to detect structural deformation of the entire superstructure at the same time providing this data quickly enough to be acted upon. The major concern during the bridge move was to limit stresses in the concrete girders and deck, as well as controlling structural twist. In order to accomplish these goal three different types of instruments were used. Strain Gauges were embedded in the precast girders and concrete deck at various locations to monitor strains. Optical Survey targets were mounted around the perimeter of the bridge to monitor structural deformations. A custom wireline system was installed at final bearing points and SPMT lift points to monitor structural twist.

Concrete embedded vibrating wire strain gauges, Model 4200 manufactured by Geokon were used to provide continuous static strain measurements during movement. Strain gauges were embedded at locations shown below in the girders and deck were maximum stress due to lifting were predicted. Gauges were located in the bottom of girders 3 and 8 at the midspan and near the surface of the concrete deck directly above the SPMT lift point on girders 4 and 7. Data from the strain gauges was collected every 2 mins during the weekend move, and was transmitted wirelessly to a project website in real-time for evaluation by key stakeholders. In order to correct for apparent strain caused by thermal expansion, an empirical temperature correction factor was developed by monitoring the apparent temperature induced strains the week preceding the move. Temperature variations in the bottom of the girder were limited to approximately 2 degree C during the move and did not significantly affect the measured strain, however the strain gauges located near the surface of the deck experience on average a 14 degree C diurnal cycle which could have entirely masked the strains induced by lifting and movement.

Optical reflective prisms were placed in pairs above the centerline of each fascia girder at the following positions, centerline of both final bearings, centerline of both SMPT bearings, and one at bridge mid span as shown below. The prisms were located so that they could be monitored during all stages of the lift, and were used to establish pre-lift (or as cast), post-lift, pre-set and final geometries of the super structure. A one arc second theodolite was used to establish absolute coordinated of the prisms at each stage of the

move. Throughout the move these measurements were compared real-time to the pre-lift measurements, allowing progressive deformation monitoring during the move. Additionally the optical prisms were used to ensure that no twist was introduced as the superstructure was set on the permanent bearings.

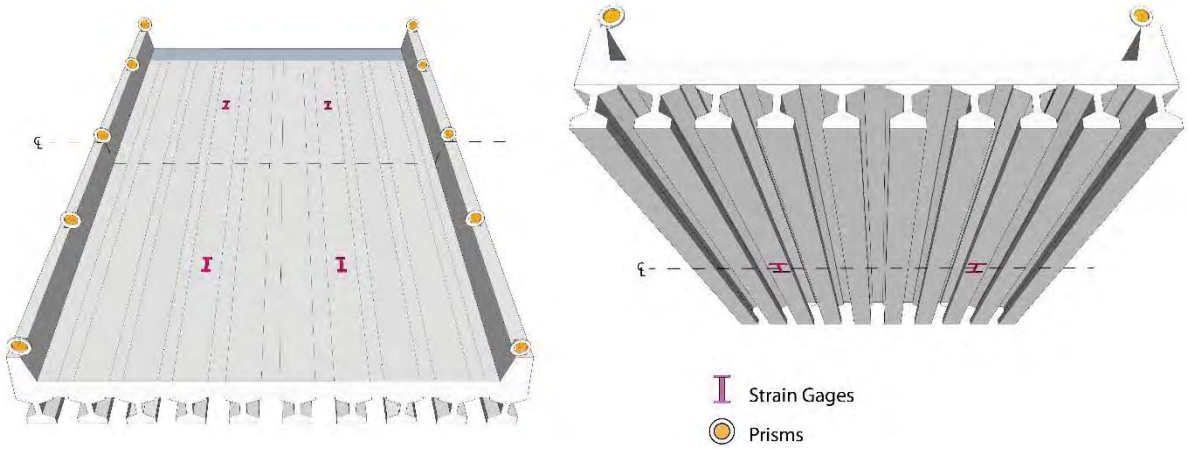

Figure 1 Layout of Optical Survey Prisms and Concrete Strain Gauges

In addition to the static strain and geometric deformation measurement made throughout the move. Two independent electrified wireline system were installed on the superstructure. One wireline system measuring deflection at the corners of the final bearing centerline and one at the SPMT bearing centerlines. These wireline systems were installed so that during lifting and movement, uneven settlement of any corner relative to the other three greater than the limit will result in the upper wires contacting the middle one, completing a circuit resulting in a siren and strobe light energizing. The limit of allowable twist was 3.5 inches measured from the final bearings, and 2.5 inches measure from the SMPT bearing line. This part of the system was integral to the move as it was the one system that was providing dynamic measurements.

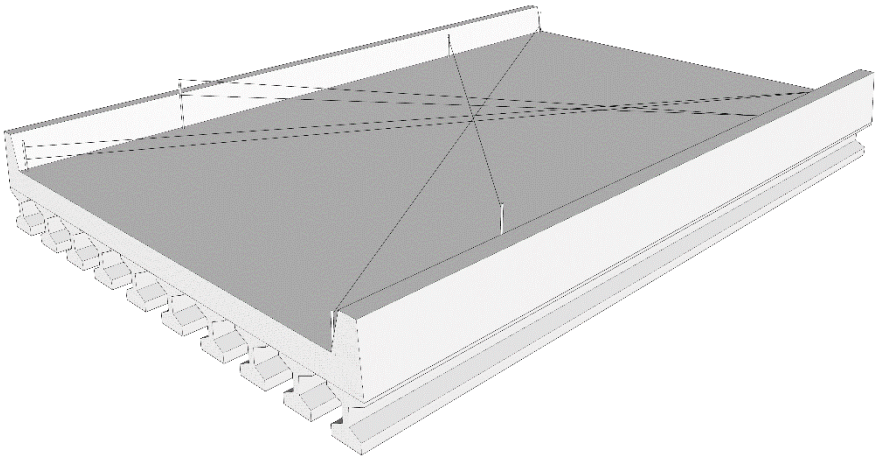

Figure 2 Layout of Wireline monitoring system

The Superstructure monitoring program allowed the movement to happened as schedule and provide direct measurement of stress and deformation during each stage. Early concern about deferential movement due to the SPMTs transiting from unpaved area to the paved area were quickly dispelled with real time review of the data. The optical survey prisms allowed the contractor to monitor the camber and calculate structural twist at multiple stages during the move. Wire lines provided "always on" measurements and were not limited to the defined reading windows of the other instruments, as such they confirmed geometric tolerances established by the bridge designer were not exceeded at any time. The strain gauges installed within the four pre-stressed girders, were consistent and showed less change in stress then predicted. Most of the Strain Gauges installed within the deck showed strain near expected values, although a couple indicated strains higher than anticipated, which was attributed to tension crack developing at the surface of the concrete deck. It is the author's recommendation to consider including additional strain gauges in areas where surface tension crack could develop, to ensure representational data is collected. At the completion of movement and final setting of the superstructures the measured strains in the deck and girders were approximately equally to the initial strains before the superstructure had been lifted from the temporary formwork used for prefabrication. These measurements provided reassurance that no unanticipated strains had been introduced due to movement.

BRIDGE PIER MOVEMENT AND FORCES DURING SLIDE-IN

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INTRODUCTION

Two Accelerated Bridge Construction (ABC) methodologies that reduce traffic interruptions typically under 12 hours are: 'Slide-in' and 'SPMT-move'. In the Slide-in, the bridge is constructed adjacent to the existing structure and laterally slid in position following quick removal of the replacement structure. In the SPMT-move, the bridge is constructed off-site, often in a staging area and transported by special vehicles and placed in position after removal of the replacement bridge. Two Slide-in projects have been completed in Michigan and another one is being implemented during 2015-2016. The first project consisted of two bridges that were designed to be pulled in place. For pulling, the superstructure is constructed on sliding girders that move on railing girders. During the sliding operation, pulling forces were applied to the railing girders. The sliding girders are supported by temporary bents; thus, the substructure was unaffected by the sliding forces. Once in alignment with the road, the superstructure needed to be lifted at each corner to place the superstructure on permanent bearings. The second Slidein project consisted of one 2-span bridge designed to be pushed in sliding tracks on to the permanent bearings. In this case, the superstructure was slid directly on the bearings installed on the substructure.

The sliding forces were directly applied to the superstructure and the friction forces were transferred to the substructure. Quantification of loads acting on the substructure during the slide is necessary to understand the system response, and to develop recommendations towards standardizing Slide-in. Hence, substructure movement was monitored during the second Slide-in project, and an analysis model was developed to calculate the forces acting on the substructure based on the field measurements (1). This article presents the monitoring of bridge pier movement and the model to calculate sliding forces.

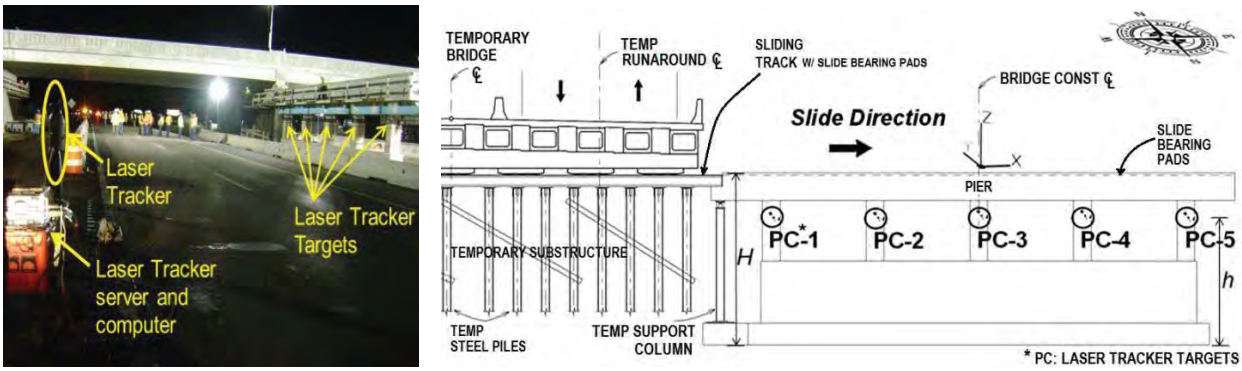
QUANTIFICATION OF LOADS

Slide-in is new and the loads acting on permanent substructure during the sliding operation are not well understood. Using load cells to quantify horizontal loads acting on the permanent substructure during the slide is impractical because the loads are primarily created from frictional resistance. For the 2-span

bridge, the abutments were supported on piles while the pier was supported on a spread footing (Figure 1). The sliding mechanism consisted of sliding tracks with slide bearing pads on temporary substructure, which provided jacking support and lateral restraint to the superstructure while on temporary structure; whereas on the permanent substructure, the sliding mechanism consisted of slide bearing pads installed directly on the substructure without lateral restraint to the superstructure. A quick analysis indicated measurable movements of the pier when the superstructure was being moved onto it from the temporary substructure. Therefore, the pier was instrumented with high precision crystal targets to measure the movements in three perpendicular directions using a non-contact high-precision laser equipment, the Laser Tracker. Then, the forces acting on the pier were calculated using soil-structure interaction, the force-deformation relations, and the field measurements.

MONITORING PIER DEFORMATIONS

Laser Tracker and was placed at the site with a view of all targets on the pier but away from the construction zone and about 150ft (45.7m) away from the targets (Figure 1a). A laptop controlled the Laser Tracker and recorded the displacements. The data collection process was programmed to direct the laser beam to each target sequentially during each measurement cycle. During the sliding operation, the automated process made measurements in small time intervals. From the measured data,displacement of each target was obtained in X, Y, and Z directions with respect to a structure specific coordinate system (Figure 1b). The measured displacements in the slide direction (X direction) and transverse direction to slide (Y direction) showed a trend of positive displacement. These measured displacements associates to the resistance to sliding forces developed when the superstructure got stuck on the sliding track. The sliding operation was stopped temporarily in several instances for lubricating the sidewalls of the sliding track to reduce friction. With the superstructure stuck, forces generated by the pushing jacks were transferred to the pier. The measured displacements in vertical direction (Z direction) at each target showed a trend of increase in the settlement as the bridge moved on to the pier. This trend associates to the increase in load on the pier as the superstructure moved on to the substructure from temporary supports.



(a) (b)

Figure 1. (a) Sliding bridge and Laser Tracker at site, and (b) coordinate system and targets

ANALYSIS MODEL TO CALCULATE FORCES ACTING ON THE PIER DURING THE SLIDE

The recorded displacements were analyzed and filtered to 18 data sets for plotting the displaced shape of the pier and calculate resultant displacements and rotations. The 18 data sets were selected based on the displacement amplitudes, associated with the superstructure position during the move. The instances of operational complications during the slide operation were also taken in account for selecting the data sets. In developing the analysis model, the pier was considered as a rigid body. Rigid body displacements in X, Y, and Z directions (Δx, Δy, and Δz) at the centroid of the pier were calculated for the selected data sets. As shown in Eq.1, Δx, Δy, and Δz are the resultants of pier translations in X, Y, and Z directions (δx, δy, and δz) and rotations about X- and Y- axes (θx, and θy). The "±" sign in Eq.1 is used to account for the directions of the pier rotations about X and Y-axes. For the analysis model, clockwise rotations are considered positive. The direction of pier rotation for each data set was identified from the displaced shape of the pier cap at respective observation.

$$\begin{aligned} \Delta_x &= \delta_x \pm \theta_y h \\ \Delta_y &= \delta_y \pm \theta_x h \\ \Delta_z &= \delta_z \end{aligned} \qquad [1]$$

All measured displacements were assumed to be due to the elasticity of the soil. Soil below and around the pier foundation provided the resistance to pier translation and rotation. The forces acting on the substructure during the slide operation were back calculated from the force-deformation (stiffness) relationship of the elastic foundation. The force-deformation relationship of a six degrees of freedom system in terms of associated stiffness coefficients (K), translations (d), and rotations (θ) was utilized.

Incorporating the uncoupled translations and rotations in the force-deformation relationship and rearranging, Eq.2 was derived to calculate the forces acting on the pier in X, Y, and Z directions.

$$F_x = \frac{\Delta_x}{\left(1/K_x \pm Hh/K_{\theta y} \right)}; \quad F_y = \frac{\Delta_y}{\left(1/K_y \pm Hh/K_{\theta x} \right)}; \quad F_z = K_z \Delta_z \qquad [2]$$

where, K_x, K_y, and K_z are translational stiffness; $K_{\theta x}$ and $K_{\theta y}$ are rotational stiffness; H is the moment arm of rotation of the pier foundation (height of the pier is considered as the moment arm); h is the distance from the base of the pier foundation to the location of targets on the pier (all targets on the pier were at approximately equal heights; thus, the average distance from the footing centroid to the targets was used for h).

The elastic settlement formulation of a foundation under a uniform pressure was utilized from National

Cooperative Highway Research Program (NCHRP) research report 343 (2) to calculate the translational stiffness (K_x, K_y, and K_z). The foundation rotation about X and Y axes develops a linear pressure distribution on the soil. Therefore, the settlement expressions for the shallow foundations subjected to eccentric loading were used from Algin (3). Based on these expressions, the rotational stiffness expressions about X-axis and Y-axis ($K_{\theta x}$ and $K_{\theta y}$) were derived. The geometric parameters of the pier foundation and the properties of soil below the foundation were utilized to calculate the rotational stiffness. Soil properties were obtained from the boring logs near the pier. The stiffness coefficients are functions of soil modulus of elasticity, *E*. Numerous methodologies are presented in literature for estimating *E*. The methodology utilized in the analysis model was obtained from NCHRP (2).

For the 18 data sets selected from field measurements, a range of force components acting on the pier was calculated from the stiffness coefficients. The variation between the calculated lower and upper bound forces was due to the range of estimated soil modulus of elasticity (E_{min} and E_{max}) values.

CONCLUSIONS AND RECOMMENDATIONS

Specific conclusions and recommendations derived from the monitoring and analysis are the following:

- During the move, forces and displacements on the pier were anticipated only in the sliding direction (X direction) from frictional forces, and in the vertical direction (Z direction) from the superstructure weight. Analysis showed that a force couple was developed rotating the superstructure (yaw) and pushing the substructure in the transverse direction to slide (Y direction).

- Transverse displacements up to 0.6in (15mm) were measured at the bridge pier. Implementing the analysis framework, the associated force was estimated between 158kips (703kN) and 357kips (1588kN). The force estimated in the slide direction was between 294kips (1308kN) and 313kips (1392kN).
- The analysis indicted that the transverse deformations is the result of superstructure rotation under a couple developed on the sliding surfaces from the difference between pushing forces and unequal of friction forces. The unequal friction forces were due to the differential friction coefficient between the sliding surfaces. Further, it is not realistic to expect equal sliding friction force developing on the sliding surfaces. The unequal friction coefficient generates unbalanced resistance that forms a couple forcing the superstructure off the sliding alignment.
- Sliding tracks are not sufficient to keep the superstructure aligned. Monitoring forces and displacements need to be an essential component of the slide operations.

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G-4: LATERAL SLIDE CASE STUDIES II

SUBSTRUCTURE CONSIDERATIONS FOR SUCCESSFUL ACCELERATED BRIDGE REPLACEMENT PROJECTS

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ABSTRACT

Accelerated bridge replacement techniques have the capability of reducing traffic closures to mere hours compared to traditional bridge replacement projects which may require lane closures for many months or even years.

In order to realize the full benefit of accelerated bridge replacement, serious consideration must be given to the existing substructure - its structural capacity, existing condition and the options available to repair and/or extend its service life. Within the context of a superstructure replacement project, it is a reasonable objective that substructures are rehabilitated such as to achieve a service life which meets or exceeds the service life of the new bridge deck or superstructure.

INTRODUCTION

Accelerated bridge construction, and accelerated bridge replacement in particular, have been successfully utilized for at least 15 years and have been accepted as an advanced bridge repair technique. As the bridge stock continues to age, accelerated bridge replacement is increasingly being utilized.

The accelerated bridge replacement technique often hinges on the use of prefabricated superstructure components or using an entire superstructure that is built off site. These components can be placed one section at a time overnight, such as using individual precast deck panels, or the entire superstructure can be replaced over a weekend using self-propelled modular transport (SPMT) units or other heavy lifting techniques (1).

The benefits of accelerated bridge construction may include items such as:

- Reduced construction time
- Reduced environmental impact
- Reduced road closures

- Reduced lane restrictions
- Reduced impact on the travelling public
- Improved community relations
- Increased worker and the public safety
- Increased construction quality (2).

While the public may be focused on the high profile superstructure lift, bridge professionals understand that accelerated construction success is dependent on many other factors, one consideration being the reuse of existing abutments and piers. If the existing substructure can be reused as is, or rehabilitated concurrently with other construction operations, a substantial savings in schedule duration, project cost or both can be achieved.

One aspect of the decision to reuse existing abutments and piers are their current condition and estimated residual service life. Ideally, the substructure (as is or rehabilitated) should meet or exceed the service life of the new bridge deck or superstructure and not require any significant premature repairs, rehabilitation or in the worse scenario replacement.

SUBSTRUCTURE EVALUATION

A general review of the substructure normally verifies that the existing components are compatible with the replaced superstructure. This process will expose limitations in the approaches, foundations, load carrying capacity or substructure dimension if for example the new bridge is to be widened.

On major rehabilitation projects such as superstructure replacement, it is prudent that the existing substructure undergoes a condition assessment and environmental characterization to understand its current condition. A typical assessment may include actions such as visual inspection, confirmation of as-built reinforcement placement via ground penetrating radar, confirmation of concrete compressive strength, petrographic examination to detect concrete abnormalities such as improper aid voids or reactive aggregates, concrete cover over the reinforcing steel, chloride ion profile, carbonation depth, mapping of cracks, spalled concrete, and concrete delaminations, corrosion potential mapping, and corrosion rate.

This information will provide a thorough overview of the current condition of the structure. In addition, service life modeling may also be warranted. A service life analysis can be accomplished by gaining an understanding of the structure’s natural environmental exposure conditions compared with the structure’s innate resistance to corrosion and concrete degradation (3).

SUBSTRUCTURE REHABILITATION

In most cases, an aged substructure can be rehabilitated even if it has been exposed to corrosive conditions. Chloride contamination from deicing salts is common in northern climates. Substructures on marine bridges often suffer from corrosion due to chlorides as well. Historically, corrosion due to concrete carbonation is not a major concern although it should be tested during the investigative phase.

Substructure deterioration caused by deicing chemical exposure can often be isolated to specific weak points in the structure such as improperly maintained expansion joints, construction (cold) joints in concrete or concrete cracks whereby soluble chloride ions from deicing salt penetrates the weak point and contaminates the substructure. An example of this phenomenon is when pier caps under a failed expansion joint are corrosion-damaged and pier caps underneath maintained expansion joints or location where there is no joint at all are in good condition.

Chloride-contamination can be more general in nature if the substructure has plowed snow pushed upon it or splashing from passing traffic leaves salt on the entire concrete surface. General contamination can also occur in marine environments to some degree by exposure to the tidal zone and airborne salts.

Once the condition assessment is complete, and a determination that substructure rehabilitation is the preferred option in terms of schedule or cost, a repair and protection strategy can be developed. There are several corrosion mitigation technologies available that have been deemed cost effective when "applied to the appropriate bridges at the appropriate time" (4). Answering two fundamental questions can point the bridge engineer in the right direction when developing a substructure rehabilitation strategy.

A. Is the concrete damage localized or widespread?
B. What is the degree of risk for future corrosion?

If the answer to Question A is that the structure suffers from localized concrete damage, then a targeted concrete patch repair strategy should be considered. An excellent reference for completing concrete repairs on corrosion damaged structures is available from the International Concrete Repair Institute (ICRI) (5). If the answer is that the concrete damage is more widespread across the entire substructure, or within a specific structural element, then a global protection plan may be necessary. If the capacity of the structure has been compromised, structural strengthening may be required. This could be accomplished with large area concrete replacement with supplemental surface-applied FRP strengthening or by utilizing a reinforced concrete encasement where the section is built out with additional reinforcing steel. As

indicated on Figure 1, as the as the percentage of surface area with concrete damage increases, the likelihood of using reinforced encasements or structural replacement increases.

The answer to Question B regarding corrosion risk lies within the information gathered during condition assessment. For this purpose, corrosion risk is defined as the percentage of area with concrete damage and high corrosion potentials as measured by a reference electrode. As the risk of future corrosion increases, it is more likely to move from a strategy of targeted protection to globally protecting large areas of the structure.

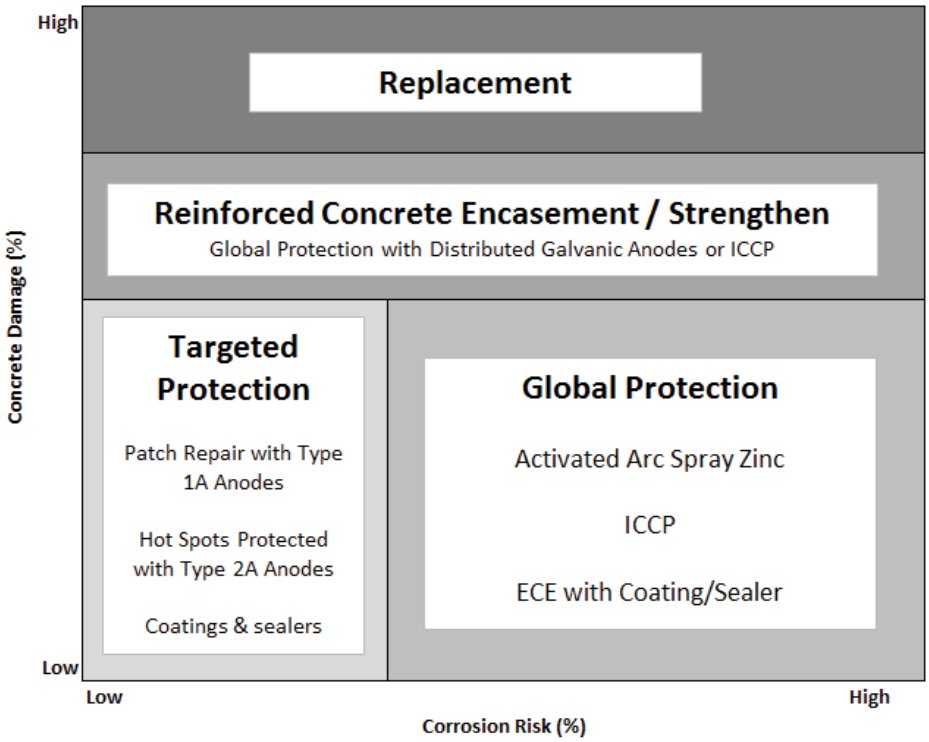

Figure 1: Matrix of Concrete Rehabilitation Strategies

CORROSION REPAIR STRATEGIES

The following information details potential concrete rehabilitation strategies that incorporate various corrosion mitigation technologies. An overview of cost effective electrochemical corrosion mitigation technologies can be found in the FHWA Bridge Preservation Guide (4) and the ICRI Guide for Electrochemical Techniques for Corrosion Mitigation (6).

Patch Repair with Targeted Protection

When corrosion damage is low and the risk of future corrosion is low, then a Patch Repair with Targeted Protection strategy can be considered. This approach would include concrete repair by ICRI Guidelines (5) with Type 1A Embedded Galvanic Anodes. Sound concrete areas with high corrosion potentials (hot spots) would be protected with Type 2A Embedded Galvanic Anodes (7). If the structure remains in a corrosive environment then it can be coated or sealed for more general protection.

Patch Repair with Global Protection

If corrosion damage is low but the risk of future corrosion is high, then a strategy of Patch Repairs with Global Protection should be considered.. Examples of global protection include electrochemical chloride extraction (ECE), activated arc spray zinc galvanic protection and impressed current cathodic protection.

Reinforced Concrete Encasement with Cathodic Protection

For structures with more substantial concrete and corrosion damage, a reinforced concrete encasement containing additional reinforcing steel provides a one-step repair, corrosion protection and strengthening option. This technique can be accomplished on abutments, columns or pier caps.

Figure 2: Pier Cap Repair Using Reinforced Concrete Encasement with Distributed Galvanic Anodes, George Wade Memorial Bridge Rehabilitation, 2011

To provide cathodic protection to both the existing corroded steel and the new reinforcing in the encasement, distributed galvanic anodes or impressed current cathodic protection are an important consideration.

Replacement

When structures are substantially damaged and deemed impractical or cost-prohibitive to repair, a full structural replacement is an appropriate option. Life cycle considerations should be taken into account in the design of the replacement substructure including concrete mix design, type of reinforcing, and cathodic prevention.

EXAMPLE PROJECTS

There are many examples of Accelerated Bridge Replacement projects that reuse existing substructures to speed the schedule or extend project budgets. We will provide two examples below:

I-480 Viaduct, Omaha, Nebraska, USA

In the summer of 2002, the Nebraska DOR undertook a large rehabilitation project on a 1.5 mile elevated section of Interstate 480 in downtown Omaha. Constructed in the early 1970's, the viaduct had experienced significant chloride induced corrosion and deterioration to the concrete deck and substructure.

The rehabilitation project included a complete replacement of the concrete deck but the long-term performance of the repair substructure was a concern. After the corrosion investigation was completed on the 66 hammerhead piers, it was determined that 23 were in a low concrete damage, high corrosion potential condition. The repair strategy for these piers was Patch Repair with Global Protection using Norcure electrochemical chloride extraction, a technique that removes chloride ions from concrete and restores the rebar to a passive condition.

Thirty-two of the piers were determined to have low concrete damage and low corrosion risk and as such utilized Patch Repair with Targeted protection using Type 1A embedded galvanic anodes to extend the life of the patch repairs.

Since the hammerhead piers were to remain in a corrosive environment, all 66 were protected from future chloride contamination with a chloride-resistant coating.

Figure 3: Bridge Deck Replacement

Figure 4: Removal of Damaged Concrete

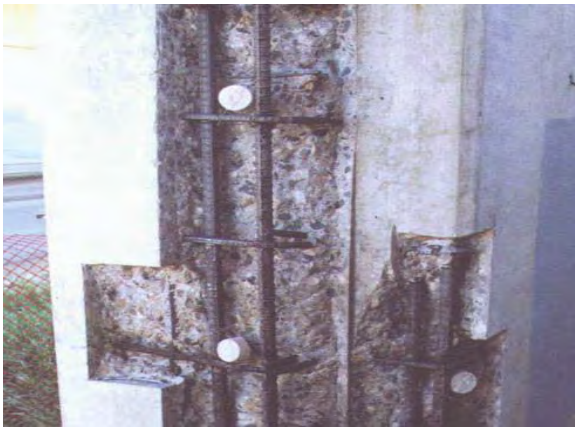

Figure 5 Patch Repair with Type 1A Anodes

Figure 6: Concrete Repairs

Figure 7: Temporary Anode Placement

Figure 8: Temporary Electrolyte Installation

Figure 9: Norcure ECE Treatment in Progress

Figure 10: Replacement of Drains and Coating Application

Figure 11: Completed I-480 Viaduct Rehabilitation

Highway 417 Island Park Bridge, Ottawa, Ontario, Canada

In August 2007, the Ontario Ministry of Transportation (MTO) completed a rapid lift replacement of the Island Park Bridge which carries Highway 417 in Ottawa. At 150,000 vehicles per day, the structure is on a heavily used artery in the national capitol.

The project design was to rapidly remove and replace the 48-year old eastbound and westbound superstructures while reusing the existing (rehabilitated) concrete abutments. The new prefabricated superstructures, consisting of steel girders and concrete decks, were built on a site adjacent to the Island Park Bridge.

While the new superstructures were under construction, the concrete abutments were rehabilitated. The abutments were substantially damaged with high risk of future corrosion. The engineers selected a Reinforced Concrete Encasement with Cathodic Protection strategy for the abutments. This was accomplished by re-facing the abutment walls with a structural overlay containing additional reinforcing steel. To protect the reinforcing steel in the existing chloride-contaminated concrete, Galvanode DAS distributed galvanic anodes were placed in the overbuilt section.

Both bridges were replaced in a 16 hour period from Saturday, August 11th at 8:00pm until the bridge was reopened Sunday August 12th at 12:00pm. SPMT units were used to rapidly remove and replace the superstructures.

The Island Park Bridge replacement was the first accelerated bridge replacement project completed in Canada. This successful accelerated bridge replacement project was estimated to have saved $2.4 million and 2 years compared to conventional techniques. Due to the benefits of tis accelerated bridge replacement project, MTO has used this approach on 4 additional bridge replacement projects.

Figure 12: Construction of New Prefabricated Superstructures

Figure 13: Condition of Existing Abutments

Figure 14: Re-facing of Existing Abutments

Figure 15: Distributed Galvanic Anodes and Additional Reinforcing Steel

Figure 16: SPMT Moving a New Prefabricated Superstructure

Figure 17 New Superstructure In-place

To see a complete detailed video of the Island Park Bridge Rehabilitation project, view the project video courtesy of McCormick Rankin Corporation https://www.youtube.com/watch?v=9JZ02f-SmAw (8)

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LARPENTEUR BRIDGE - SLIDE-IN BRIDGE CONSTRUCTION OF A TWO-SPAN BRIDGE

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FOREWARD AND ACKNOWLEDGEMENTS

The Larpenteur Avenue bridge was the first bridge in Minnesota to use the Accelerated Bridge Construction (ABC) technique called Slide-In Bridge Construction (SIBC). The Minnesota Department of Transportation (MnDOT) used this opportunity to incorporate the Federal Highway Administration's (FHWA) initiative for accelerated bridge construction.

The Design Build project team consisted of Ames Construction, Alliant Engineering and Michael Baker International (Ames Team). James Ronning provided the heavy lifting engineering.

PROJECT OVERVIEW

The Larpenteur Avenue bridge replacement was part of a $98 million Design-Build project which included nine bridge replacements (three bridges over I-35E, three I-35E northbound bridges, and three I-35E southbound bridges), roadway widening, and other improvements along the interstate corridor. Having received the highest technical score, the project was awarded to the Ames Team based on a "best value" bidding system.

A lateral bridge slide was not required by the contract, but was implemented as an Alternative Technical Concept (ATC) to reduce closure time and project schedule. The Larpenteur bridge is the first bridge in Minnesota to utilize the SIBC technique which is emphasized in the Federal Highway Administration's Every Day Counts initiative. By using SIBC for the Larpenteur Avenue Bridge, the construction time and overall closure duration was reduced from 110 days to 47 days.

Vertical Geometry Constraints

The superstructure was built north of the final location in a temporary staging area, and slid approximately 84 feet into final position. The two major items that dictated the location of the bridge staging area were:

- Sensitive utilities south of the bridge that could not be relocated, and
- Reduced vertical clearance over I-35E north of the bridge.

I-35E has a profile grade that slopes upward to the north. The bridge needed to be constructed in the temporary staging area at the final proposed elevations causing a reduced vertical clearance. When comparing alternatives, the temporary reduction in clearance vs. costly utility relocations was determined to be the best.

Bridge Geometry

The Larpenteur Avenue bridge traverses Interstate I-35E in Saint Paul, Minnesota. The bridge is a two-span prestressed concrete girder bridge with a 9-inch composite concrete deck. Each span is 91’-9” with seven girders at 11’-4” spacing and an out-to-out bridge width of 75’-10”. The bridge substructure consisted of a reinforced concrete pier and reinforced concrete semi-integral abutments. The substructures were all founded on spread footings.

SIBC Detailing

MnDOT’s standard details required a few modifications in order to accommodate the SIBC construction. The wingwalls were constructed with block-outs to allow the proposed superstructure to slide past, requiring an additional concrete closure pour after the slide was completed. Modifications were also made to the owner’s standard semi-integral abutment diaphragm. The Ames Team incorporated a split diaphragm at the pier to allow the bridge spans to continue to act as simple spans. The typical bearing details were adjusted to incorporate the slide. The standard MnDOT curved plate was removed, and only bearing plates and steel reinforced elastomeric pads were used. Anchor bolts were used at the pier to provide fixity, however the slide required a level surface at the top of the pier cap. To install the anchor bolts, ducts that were deep enough for the entire bolt to fit in, were provided at the anchor locations. After the slide, the bolts were lifted out of the ducts through the slotted bearing plates, set to the required projection length and then grouted into place.

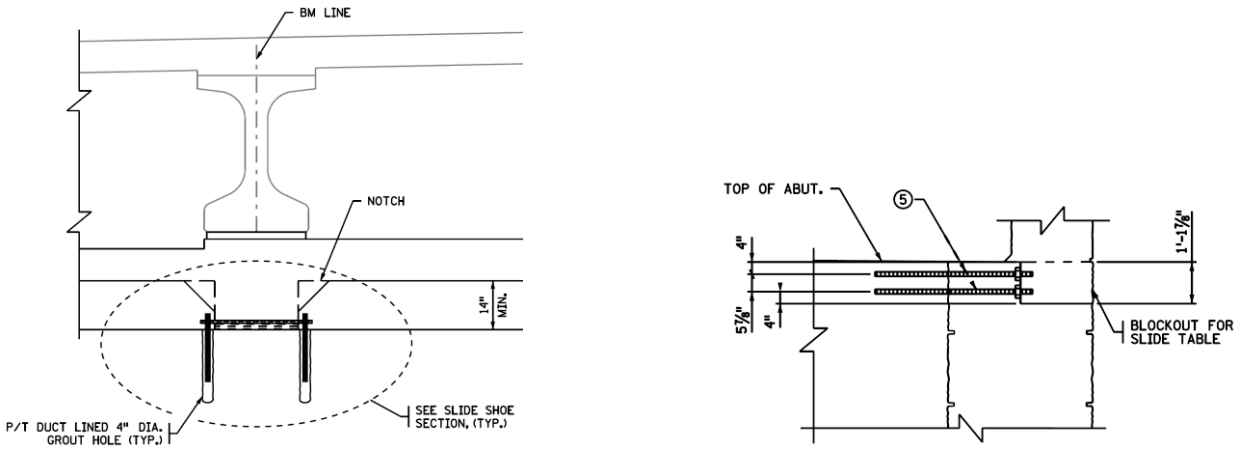

Figure 2: SIBC Details

SIBC Design

Temporary steel H-pile slide tables were used to support the superstructure in the temporary position adjacent to the existing bridge. While the existing bridge remained in service, the slide tables were erected, and the proposed bridge superstructure was constructed in the temporary position. The slide tables were anchored into the permanent abutments and pier cap to provide additional lateral capacity. This also prevented differential movement between the sliding surface of the temporary supports and the permanent substructure elements.

After the deck was poured on the proposed superstructure, the existing bridge was closed, demolished, and the proposed substructures were constructed. After completion of the substructure and proper cure of the deck was achieved, the superstructure was ready to be moved into its permanent position.

The 4 million pound multi-span bridge was slid into place using a system of hydraulic jacks; one at each abutment and two at the pier. The jacks pushed off a frame system that was connected to the temporary slide tables by a series of pins and rails. For the bridge slide galvanized sole plates were embedded at each bearing location under the end concrete diaphragms. These sole plates rested on skid shoes comprised of neoprene and Teflon pads. The neoprene acted as a shock absorber while the Teflon provided a low friction slide media. The slide track was lined with polished stainless steel strips, providing a low friction surface for the Teflon to slide upon. As the jacks reacted off the pin and guide rail system, the skid shoes moved across the stainless steel tracks that were placed along the permanent pier and abutments.

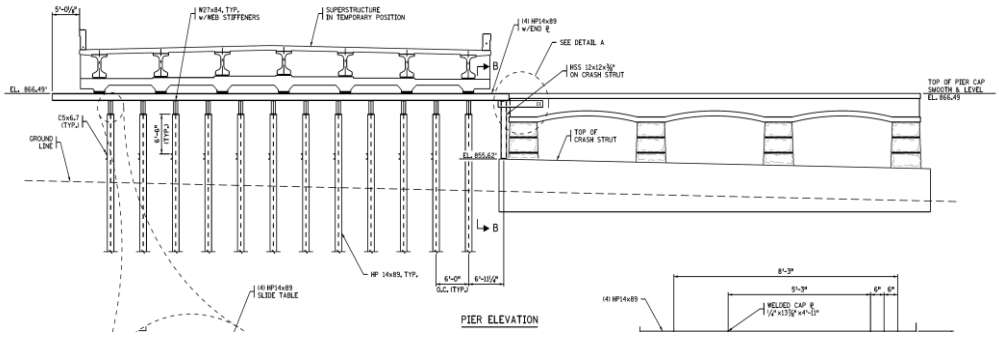

Figure 3: Temporary Pier Support

SIBC Execution

The bridge was slid into place using two night closures of I-35E. There were some complications with the slide due to lateral bridge movement. This caused some delay due to additional retrofitting and geometric adjustments, but was remedied by restraining lateral movement against the partially constructed wingwalls.

CONCLUSION

By using SIBC the construction closure duration of Larpenteur Avenue was reduced from 110 days to only 47 days. This accelerated schedule reduced the amount of days the traveling public would have to detour and expedited local access to stakeholders and businesses affected by the Larpenteur bridge closure. The project success is attributed to the coordination and cooperation between the Ames Team and the Minnesota Department of Transportation. MnDOT now has experience with SBIC, and can look for ways to incorporate this ABC technique to reduce construction schedules and minimize impacts to the public.

GOING THE DISTANCE: THE LATERAL SLIDING OF FOUR RAILWAY BRIDGES AT WEST TORONTO DIAMOND IN TORONTO

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ABSTRACT

This paper describes how Accelerated Bridge Construction (ABC) techniques were used for the construction of four railway bridges for the West Toronto Diamond Rail to Rail Grade Separation project in Ontario, Canada. The project consisted of eliminating at-grade diamond crossings of the Metrolinx Kitchener Corridor (corridor formally owned by the Canadian National Railway (CN)) and the Canadian Pacific Railway (CP) tracks in the Junction area of Toronto, an area which takes its name from the confluence of these railways. Since the 1880's, rail traffic here has been constrained by these diamond crossings involving the CN and CP mainlines and a CP Wye track.

The project involved relocating the Metrolinx tracks below the CP tracks, while at the same time maintaining all rail operations with a minimum of interruption to the Railways' activities. The site is physically constrained and hence, in order to accomplish this goal, Parsons incorporated ABC techniques into the design of the bridges carrying CP tracks, including the sliding of 4 mainline railway bridge spans, weighing a total of 12,600 tonnes, into their final positions up to 80m (262 ft) away from where they were constructed. Each slide occupied only a few hours, as it was powered by computerized high-speed tandem hydraulic jacks, moving these massive structures on steel / bronze slide paths, enabling the bridge spans to move quickly and continuously into position during short term possessions of the tracks.

Parsons' experience in large-scale heavy bridge slides, in conjunction with liaison with the Owner (Metrolinx) and the key Stakeholder (CP), was a key factor in the initial acceptance and success of this component of the overall project. ABC techniques proved to not only provide a durable and technically acceptable product for these railway bridges, but allowed the structures to be constructed within operational windows acceptable to the Railways. This paper will review each stage of the ABC techniques from inception to their final construction and the ultimate project completion.

In addition to the lateral siding, bridge construction activities on site were accelerated by incorporating full depth precast deck slab elements, transversely connected by means of post-tensioning, and foundations consisting of a unique interlocking steel pipe pile wall system.

INTRODUCTION

The West Toronto Diamond project is owned by Metrolinx and is part of the Greater Toronto Area passenger rail network system currently being expanded to address the steady increase in transit ridership associated with a steady increase in population.

The project consisted of eliminating two existing at-grade diamond crossings of the Metrolinx Kitchener Corridor and the CP North Toronto Subdivision, with a one kilometer long grade-separated structure. The Metrolinx corridor was depressed below grade while the CP tracks remained at their current elevation and location. The CP tracks at this site are used extensively to haul transport between Toronto and Montreal, whereas the Metrolinx tracks predominantly carry passenger trains operated by Metrolinx and VIA Rail, as well as CN freight.

The project is located in one of the oldest parts of the City of Toronto, with the original construction of the railway corridors occurring in the late 1800's. The area has been built-up with both industrial and residential properties surrounding the original Railway rights of way resulting in significant property constraints.

The original at-grade crossing of the CP North Toronto and the Metrolinx Kitchener Corridor was able to satisfy the current demands of CP freight and Metrolinx passenger trains, but cannot accommodate any future capacity, as a result of increased demands for commuter trains and freight traffic, nor the proposed airport rail link, from Downtown Toronto to the Toronto International Airport. As a result of these constraints, Metrolinx undertook construction of a rail-rail grade separation at this site.

PROJECT DESCRIPTION

The project consisted of several components which required special attention due to significant amounts of interaction which generate unique challenges, but which also allowed for the potential for innovative solutions.

The project site is very congested and complicated by the presence of three property owners (CP, Metrolinx, and City of Toronto), four operating railways (CP, CN, Metrolinx and VIA Rail), extensive buried utilities including a 1200 (4 ft) diameter trunk water main, a 1500 (5 ft) diameter combined

Figure 1 – Location and Existing Site Conditions

sanitary / storm sewer, railway signals, and four fibre optic companies' services with main conduits running within the railway corridor.

The original Metrolinx Kitchener Corridor consisted of two tracks, which intersect the CP Wye track adjacent to Old Weston Road and the two CP North Toronto Mainline tracks. These CP corridors are located approximately within 500m (1650 ft) of each other. In addition to the Metrolinx and CP tracks, there are two additional wye tracks which connect the CP and Metrolinx Subdivisions and run parallel along part of the depressed corridor.

The Old Weston Road and CP Wye track converge just outside the limits of the depressed corridor. Due to the geometry of the roadway and the sweeping curve of the track, it was not feasible to separate the crossing into two structures, and a single slab bridge carrying both the road and the rail was proposed.

Slab bridges were also selected for the CP North Toronto Subdivision bridges so that the existing track centres could be maintained and future tracks could be placed on minimum track centres to minimize the width of the rail corridor at the site. The North Toronto subdivision tracks also cross the depressed corridor on a 43 degree skew which becomes less consequential with slab type structures. The proposed depressed corridor will accommodate up to four tracks on the Metrolinx Kitchener Corridor and will be grade-separated from both the CP North Toronto Subdivision tracks and the CP Wye track. The structures supporting the CP North Toronto Subdivision will also accommodate up to four tracks allowing for two additional tracks in the future.

Due to the tight vertical geometry, and staging issues associated with providing one operational Metrolinx track for the duration of the project, two separate corridors were detailed with a common centre wall. By dividing the corridor into two segments, the vertical clearances were achieved by incorporating maximum approach grades between the roadway constraints at the limits of the project, without requiring a track lift of any CP tracks, hence greatly reducing the impact to CP operations and track infrastructure.

PROJECT STAGING

The project staging evolved from the preliminary design and is a good example of how innovation such as ABC can be developed even during the detailed design phase of such projects.

Preliminary Staging

The preliminary staging for the project, undertaken by others, considered the use of multiple track diversions in order to construct the structures at the railway crossings by conventional methods. This resulted in a significant amount of rail and signal work, including a main signal bridge relocation and several complex signal and cable bungalow relocations, potential interruptions to railway operations as well as replacement of large transformers.

The scope, scale, schedule and duration of the project revolved around these many track diversions, and the works associated with the ancillary (but very significant) signals and other relocations required to effect these track diversions. In addition, the project schedule took due account of the time (potentially) required for the various Railways to effect these diversions and signal relocations, and ultimately to reinstate the entire project.

Modifications to Preliminary Staging to Incorporate ABC

Figure 2 – Original Track Configuration

Figure 3 – Final Depressed Corridor

Parsons reviewed the project with a view to determine whether or not some of these track diversions could be minimized or eliminated, as they had such a significant effect on the scope, schedule and cost of the project. As a result of our detailed review of the original preliminary engineering design for the staging of the project, Parsons was able to re-work the entire project and develop a refined staging programme which eliminated all CP track diversions. This was done by means of utilizing ABC in the form of specialist sliding technology for the construction of the CP bridges. The introduction of this single ABC component had a significant impact on the overall project, as it eliminated 6 of the 7 original track diversions.

The proposed staging utilized the sliding of all four railway structures (2 CP North Toronto & 2 CP Wye/Old Weston Road Bridges) with the bridges installed in two separate stages, one for each corridor. The revised staging plan and the proposed bridge sliding required review and acceptance from the Owner and the Railways before it could be deemed a successful innovation for this project. Considerable detail of the staging and sliding scheme was developed so that all the stakeholders could assess whether or not the proposal had merit. Ultimately, the Railways were very enthusiastic and found the advantages of the scheme to be clear and extensive. Parson's experience in other large-scale heavy bridge slides was one key factor in the Railways and Metrolinx accepting this unusual technique at the West Toronto Diamond site.

ABC COMPONENTS

Precast Bridge Components

The bridge superstructures consist of pre-cast pre-stressed full depth slabs with a maximum width of 2.4m (8 ft) to allow for conventional shipping. Table 1 provides details for each of the four bridges. Joints between slabs were detailed with shear keys with the outside perimeter and the PT ducts openings within the slab sealed with EVA foam to allow the joints to be filled with epoxy grout to ensure full contact between the precast slabs. Once all the girder slab sections were erected, assembled and adjusted, the joints were then subsequently filled, with each joint filled as part of a continuous grouting operation.

Table 1

Structure	Corridor	Span length m (ft)	Width m (ft)	Number of Precast Segments	Thickness mm (in)
North Toronto Sub	East	11.4 (37.4)	35.4 (116.1)	15	900 (35.4)
	West	11.4 (37.4)	35.4 (116.1)	15	900 (35.4)
CP Wye / OWR	East	11.4 (37.4)	86.0 (282.1)	36	800 / 600 (31.4 / 23.6)
	West	11.4 (37.4)	78.8 (258.5)	33	800 / 600 (31.4 / 23.6)

In order to provide lateral structural continuity and reinforce the slab locally at the bearings, transverse PT was installed to provide a nominal compressive stress and to allow the slabs to perform as a two way structural system. All the post-tensioning was installed and stressed prior to sliding.

In the case with the CP Wye / Old Weston Road Bridge, where the slab is constructed using two different thicknesses, the segments of each portion with a uniform thickness were epoxied and post-tensioned

Figure 4 – Precast Slab Joint Details

together, with a subsequent cast-in-place transition section and additional post-tensioning installed at the support region and made continuous over the full width of the structure. Precast approach slabs were also utilized due to the large skew angle between the track and the structure. The ends of the approach slabs were detailed with preformed holes to allow the slabs to be connected to the deck overhang using straight vertical dowels with the annular space filled with epoxy.

Lateral Sliding

The technology adopted for the bridge slides at the West Toronto Diamond site is the same group of technologies used by the British railroads, even on the high speed rail tracks carrying such trains as the Eurostar, and where their railway network is significantly more complicated than in North America. The use of similar slab structures for both crossings allowed for the same sliding technology to be used for both bridges.

Figure 5 – Tandem Jacking Arrangement and Control Setup

All four CP Wye / Old Weston Road and CP North Toronto Subdivision bridges were designed to be slid into place during short term possessions of the railway tracks. The bridges were designed to be constructed adjacent to their final locations on slide paths which coincide with the centerlines of bearings. The two CP Wye / Old Weston Road bridges weigh approximately 4500 tonnes each and were required to be slid for a distance of 80m (262 ft), and the two CP North Toronto Sub bridges weighed approximately 1800 tonnes each and were required to be slid 28m (92 ft).

The general contractor (Grascan Construction Ltd.) utilized a specialist sub-contractor, (Western Mechanical Electrical Millwright Services Ltd.) to perform the bridge sliding for the project. The sliding system for this project incorporated large high-speed tandem hydraulic jacks (HSL) controlled by computerized systems (Smart Cylinder Control) which can ensure movements and loads are properly monitored for accurate positioning and uniform movement of the structure at all times during the slide. The tandem jack arrangement allows the slide to progress at a steady rate without disruption, as one jack is always engaged. The anticipated rate of advancement for bridges utilizing this technology is 20m/hour, which translates to a four hour slide for the 80m (262 ft) long Old Weston Road structure and a two hour slide for the North Toronto Subdivision bridge.

Slide Path and Sliding Bearings

Due to tight schedule requirements of the Railways, it was determined that the bridges would be slid on their permanent bearings and secured in place after the slides without the need for subsequent jacking or remedial work to the slide path. Since the permanent bearings are to be used for the sliding, it was important to establish how the sliding mechanisms would be built into the bearing assemblies and how they interacted with the bearing slide path. Elastomeric bearings were detailed as the vertical load-supporting element and were placed within steel assemblies which could resist the large lateral loads due to the bridges providing lateral support to the tops of the walls, in addition to being able to incorporate the specific sliding surfaces required in the overall bearing assemblies.

The sliding interface used comprised of bronze on steel with an estimated breakout (static) friction of 15%, and a sliding (dynamic) friction of 8%.The bearings were designed by VSL International (VSL) and consisted of a laminated bearing with steel top and bottom plates and an external bottom bearing plate with the bronze recessed into the underside of the plate. These materials would achieve a reasonable coefficient of friction in addition to satisfying the railway requirements for acceptable materials used in the permanent bearings.

Figure 6 – Typical Bearing Layout

The slide paths consisted of a temporary portion and a permanent portion based on the pre-slide and post- slide locations of the structure. The permanent portion consisted of a series of steel plates each being approximately 2.4m (8’) long and permanently anchored into the top of the wall under the post-slide location. The plates used on the pre-slide portion of the slidepath were

anchored and temporarily removable for re-use for the other bridge slide locations. All slide path plates were machined to a very tight tolerance and a very tight tolerance was stipulated for the overall alignment of the slide path.

The bearings were guided on one of the slide paths only, with the guides located on the two outermost bearings, while the remaining bearings were unrestrained allowing the bearings to freely slide in both directions.

One of the key operations which also needed to be completed before the bridge could be opened to railway traffic involved anchoring the bearings after the bridge was slid. A unique anchoring system allowing the bearing fixation to be done by means of bolting the anchorage system to the slide path, was developed and implemented successfully.

Corridor Walls with Integrated Foundations

Each of the retaining walls comprising the depressed corridor are approximately one kilometer in length, with a maximum excavation depth of 11 metres (36 ft). The wall system for the corridor as well as the bridge foundations, sustained from the preliminary design, consists of an interlocking steel pipe piling system which was developed in Japan and was found to be virtually unknown in Ontario. It proved to have a number of advantages for the depressed corridor walls and was utilized for the bridges by means of extending the wall into the underlying dense glacial till material capable of supporting the vertical reactions of the bridge. Piles were augured out and filled with concrete over their full length. A significant advantage to this type of foundation was that it allowed for the foundations to be installed at grade during short term railway possessions with minimal disruption to the railway and only required temporary track removal where the wall was situated directly below the rails.

As a result, the only cast-in-place construction required for the bridges was for the caisson wall caps, which were installed by utilizing the temporary spans and tunnels. Precast concrete or steel caisson cap construction was considered, but was deemed to be impractical based on the accuracy achievable for the pipe-pile wall installation.

Precast and Modular Temporary Works

In order to allow for the construction of the concrete caisson caps which support the bridge slide path and bearings, a combination of precast concrete tunnels and removable steel temporary spans were utilized to allow for the construction of these elements to be done with minimal disruption to the railway. Precast concrete tunnels were utilized primarily for the caisson cap construction for the CP Wye track where temporary spans were prohibitive due to the skew of the track to the centreline of the tunnel walls as well

as along one wall at the CP North Toronto due to the close proximity of relocated utilities. Tunnels consisted of rigid frame inverted U-shaped tunnel segments with separate precast spread footings and installed during short term closures of the track. Grout tubes were provided within the footings to allow voids to be filled below the footings and reduce settlement concerns.

Figure 7 – Temporary Spans (Span in foreground removed to allow for wall construction)

Temporary single track bridge spans allowed for easier access to the caisson wall construction as well improved flexibility to minimize impacts to the railway. The temporary spans consisted of steel beam spans with a span of 7.2m (23.6 ft) and direct fixation of the timber ties. Precast concrete abutments supported on micropiles were utilized to support the temporary spans to minimize disruption and allow the construction to be completed within a short duration track block. Micropiles were installed at grade between the ties during short term track blocks with the abutments and steel span installed during a single longer term track block.

IMPACTS OF ABC ON PROJECT

Durability

In terms of durability, construction joints between precast elements have traditionally been the weak link in the system, thus reducing the overall benefits of incorporating pre-cast elements. The preferred structure from the railway was a monolithic cast-in-place slab; however, in order to minimize the thickness of the slab to meet vertical clearance requirements to the Metrolinx tracks below, a prestressed slab was necessary. In order to address the concern with joints, it has been demonstrated on many segmental concrete structures that the use of PT across the interface joint of precast construction is a well proven method to mitigate against the disadvantage of numerous joints in the deck structure and this was found acceptable to the Railway.

Motivational Impact on Project

As in the case with any large project, and in particular where ABC is not a significant component of the work, it can be difficult to perceive progress. Utilizing ABC for the bridge work on this project not only

provided motivation to the Contractor, but also to the overall project team. It also allowed for the Owner to engage interest from the general public as well as their ridership community. Public relations with both the community as well as commuters is very important to Metrolinx for large urban projects such as the West Toronto Diamond grade separation, and ABC was able to generate positive interest in the project.

Bridge Sliding

When bridge sliding was proposed for the railway structures, the appropriate technology for both the design and construction works was selected to enable safe, reliable and completely predictable performance. This is of particular importance with railway structures since the bridge slides must be successfully completed on schedule. If there is a problem, it only takes a few hours before the impact to the Railway, affects railway operations and their customers across the country.

There was considerable discussion with the Railways regarding the technical aspects of the sliding,

Figure 8 – Clearing Leading Edge of Deck

Figure 9 – View from Jacking End of Deck

together with the sequence of possible track possessions which would be required to enable this project to proceed. It was found that the provision of equipment and services necessary to execute the sliding of such structures was a relatively limited portion of a project of this scale. Hence, the implementation of the sliding of the bridges at these two CP/Metrolinx crossings should reduce the cost of the project and eliminate a number of major construction operations altogether, as well as having the potential to reduce the overall schedule, and certainly to eliminate a number of construction operations from the critical path of the schedule.

The sliding aspect of the project was verified by VSL, a specialist contractor, in both the design and preparing the Contract Documents associated with the sliding of such structures so that the design and construction activities were fully integrated and so that risk to the Owner and to the Railways was

minimized by ensuring that the designs were completely identified in terms of a specific sliding system and methodology.

In order to ensure the a successful sliding operation during each track block, a 5m (16') trial slide was incorporated into the design for each of the bridge slides which could be carried out without requiring a railway track block prior to the main scheduled slide. This allowed the Contractor to set up the full jacking system as well as calibrate the jacking system in advance of the sliding track block.

CONCLUSION

Each bridge was erected and assembled as designed, each structure was successfully slid, and the railway was reinstated within the track block provided. In addition the overall project was completed on schedule allowing for the completion and commissioning of the new airport rail express train service prior to the PanAm Games held this past summer in Toronto. ABC played a critical role in allowing Metrolinx to fulfil this significant commitment to provide this vital rail service on the Metrolinx Kitchener Corridor.

The requirement for minimizing disruption of existing traffic is paramount for the Railways, as any disruption impacts their business, their customers and has direct costs to the Railways. ABC has significant potential to assist in reducing the impacts which construction has on road and rail traffic and the public perception of construction projects in general. There are often sound economic arguments for ABC, and these reasons should dominate the decision of when to apply these techniques, particularly on major public road and highway projects, where the impacts of traffic disruption clearly can have a very large negative impact on society, the environment and the economy.

M-100 SUPERSTRUCTURE SLIDE

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ABSTRACT

The Michigan Department of Transportation (MDOT) selected URS Corporation (now AECOM) to assist with the slide-in design for the replacement of the bridge carrying M-100 over the CN/GTW railroad in Potterville, Michigan. With the existing bridge still in service, temporary steel frame abutments and a temporary road were constructed adjacent and parallel to the existing structure. The new superstructure was constructed on the temporary abutments. Traffic was shifted to the temporary bridge and after the existing bridge was demolished, new permanent full-height concrete abutments were constructed on the existing alignment. A weekend shut down of traffic allowed the new superstructure to be slid from the temporary abutments onto the permanent abutments utilizing high-capacity steel rollers and horizontal jacks.

PROJECT BACKGROUND

Located a few miles southwest of the state capitol of Lansing, Michigan, the city of Potterville is split down the middle by M-100. M-100 is a two-lane state highway that runs north/south through the city with an average daily traffic volume of approximately 5400 vehicles. On the north end of town, the Canadian National/Grand Trunk Western (CN/GTW) Railroad runs southwest to northeast. Most of the residential and business areas of town are located south of the railroad tracks, but fire and police emergency services are located north of the tracks. The entire project involved roadway improvements as well as three bridge replacements – M-100 over the CN/GTW railroad, M-100 over the Thornapple Drain, and M-100 over the Sharp Drain. This report focuses on the replacement of the M-100 bridge over the CN/GTW railroad.

The M-100 bridge is located approximately 0.8 miles north of Exit 66 off of I-69. The existing structure was a three-span structure with an overall length of 157’-0” and a clear roadway width of 40’-0”. The concrete deck was supported on rolled steel beams, which were supported by reinforced concrete counterfort abutments and steel pier bents on spread footings. The existing bridge, which was in poor condition, was constructed in 1940.

CONSTRUCTION STAGING DECISIONS

Prior to selecting AECOM as part of the design team, MDOT made the decision to utilize accelerated bridge construction (ABC) at this site. The decision was based on a number of reasons:

- The shortest detour route around the project was 2.6 miles long and includes an at-grade crossing of the railroad. However, there are approximately 40 trains per day traveling the CN/GTW railroad at this location, making this detour undesirable (see Figure 1).
- There was a strong desire to minimize user delay costs, particularly as it related to the detour alternative discussed above.
- As mentioned previously, emergency services are located on the opposite side of the railroad tracks from a majority of the population.
- MDOT and city officials did not want the bridge out of service for an extended period of time.
- Part width construction was not feasible due to the narrow width of the existing bridge, which would allow only one lane to maintain two-way traffic.
- There was adequate open space on either side of the existing bridge to build a temporary structure and utilize slide-in construction.

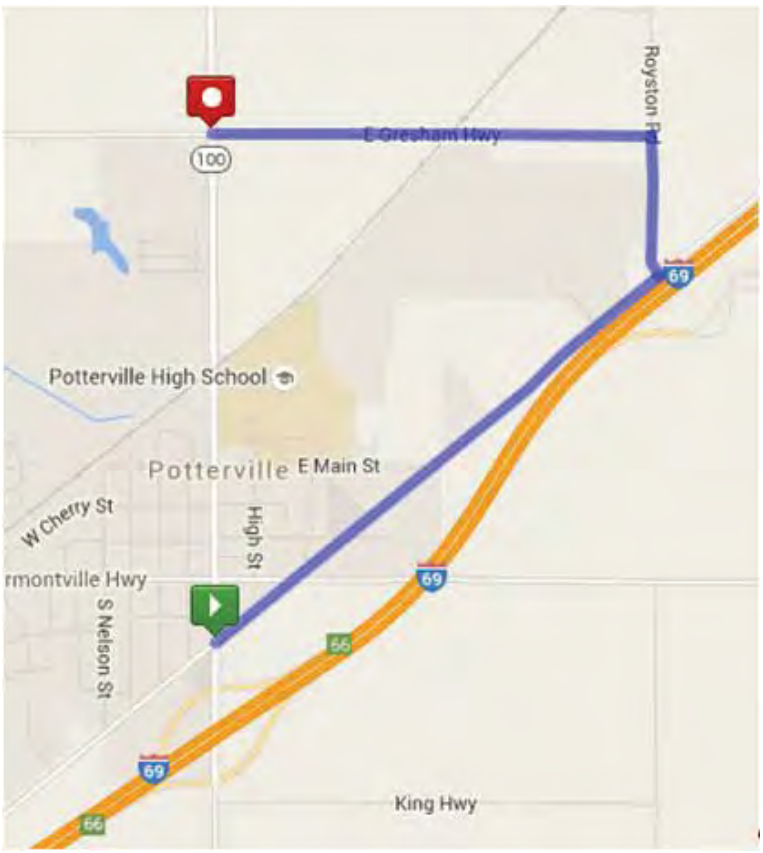

Figure 1 – Detour Route
(map provided by www.MapMyRun.com)

Sliding in the proposed superstructure versus sliding out the existing superstructure was studied at the beginning of the project. The presence of the railroad and the condition and configuration of the existing bridge were the primary factors to consider. Slide-out construction would have required temporary substructure units to be built in-line with the existing pier locations, which are 15 feet away from the centerline of the closest set of railroad tracks railroad tracks. Construction of those temporary substructure units near the tracks would have been interrupted several times per day by the multiple train movements. In addition, extending the existing pier crashwalls would have required steel sheet piling installation in order to expose the top of the existing pier footings to tie in the crashwall extensions. Installing the sheeting under the existing bridge may have been difficult work to perform. Much of the existing superstructure and substructure exhibited deterioration, which made a slide-out questionable. The proposed bridge span length located the temporary and permanent abutments outside the 25 feet railroad clearzone allowing mostly uninterrupted construction. This meant that slide-in construction would minimize conflict with railroad operations. Ultimately, MDOT decided that there was too much risk involved with slide-out construction and therefore slide-in construction was selected.

Reasonable area existed on either side of the bridge from which to stage the work. The east side was complicated by the presence of a tower carrying a high-voltage line, other overhead power lines, a buried gas line and a school. The railroad track profile was lower to the west than it was to the east. Therefore, it was decided to stage construction from the west side (see Figure 2).

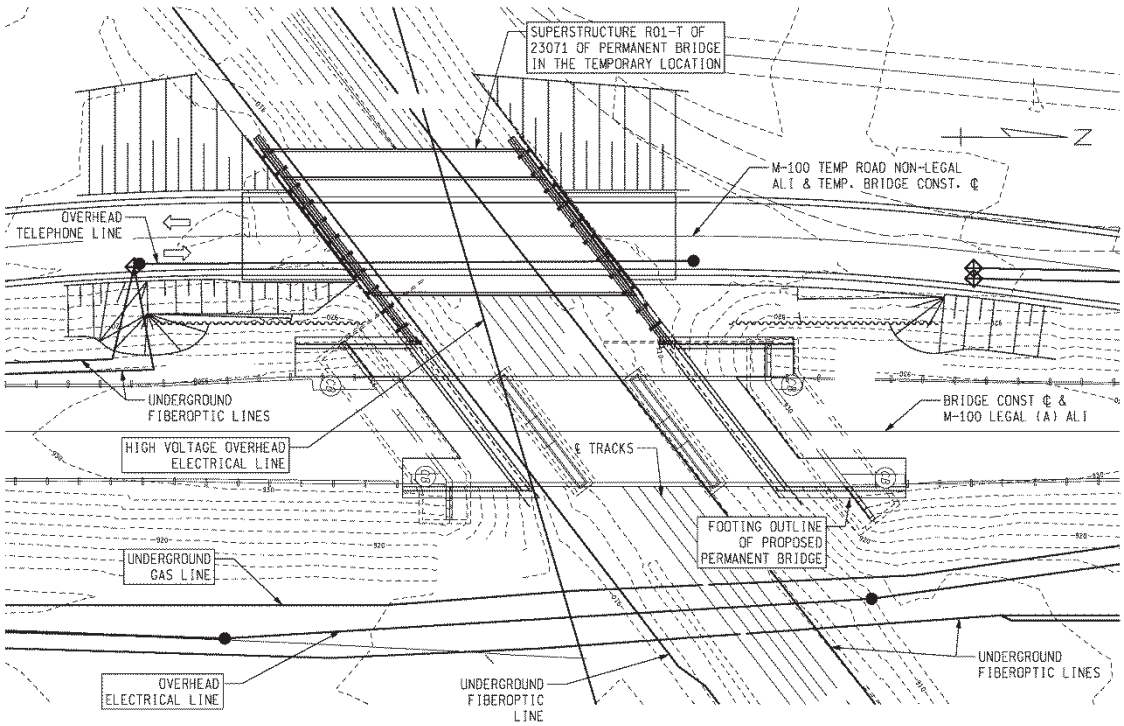

Figure 2 – Site Plan

The proposed superstructure would be constructed on a temporary alignment west of the existing bridge. A temporary road would be constructed up to the superstructure on the temporary alignment and traffic diverted to the temporary road and bridge. With traffic out of the way, the existing bridge could be demolished in its entirety and the new abutments constructed on the existing alignment. Once the new abutments were in place, the proposed superstructure will be slid into place over the course of a weekend. M-100 will only be closed for approximately 48 hours instead of the 5 to 6 month timeframe that would occur using traditional construction methods.

PROPOSED BRIDGE AND TEMPORARY ALIGNMENT DESCRIPTION

The proposed bridge spans over the railroad with a 107’-6” long single span at a 37°35’0” skew. The cast-in-place 9” thick concrete deck is supported on 40” steel plate girders which rest on full-height concrete abutments, supported on steel H-piles. The clear roadway width is 44’-0” with a 2% crown. In addition, a

10’-0” wide pedestrian path is separated from traffic by combination railing. The temporary alignment is located 76’-5” west and parallel to the existing alignment. MDOT performed the design of the new bridge as well as the roadway and traffic components of the project. AECOM designed the temporary substructure units and the slide-in components

TEMPORARY SUBSTRUCTURE DESCRIPTION

The geotechnical recommendations recommended deep foundations for both the permanent abutments and the temporary abutments. Several temporary abutment options were studied. Options included cast-in-place concrete, structural steel, driven H-pile foundations and micropiles. Ultimately, a steel frame bent was selected because it could be installed faster (no waiting for curing time) and could be recycled and/or salvaged when the project was over. The temporary abutment frames were constructed from 2 rows of 14” steel H-piles extending out of the ground to form steel bents. The piles were spaced at 8’-5” center-to-center with 4’-0” between the two rows of piles. Since the proposed permanent abutments were also supported on piles, the two substructure units would have similar stiffnesses. This helped to avoid potential differential settlements during the sliding operation. H-piles within 25 feet of the existing spread footing abutments were installed in prebored holes in order to minimize the risk of vibration induced settlements of the existing footings during pile driving operations.

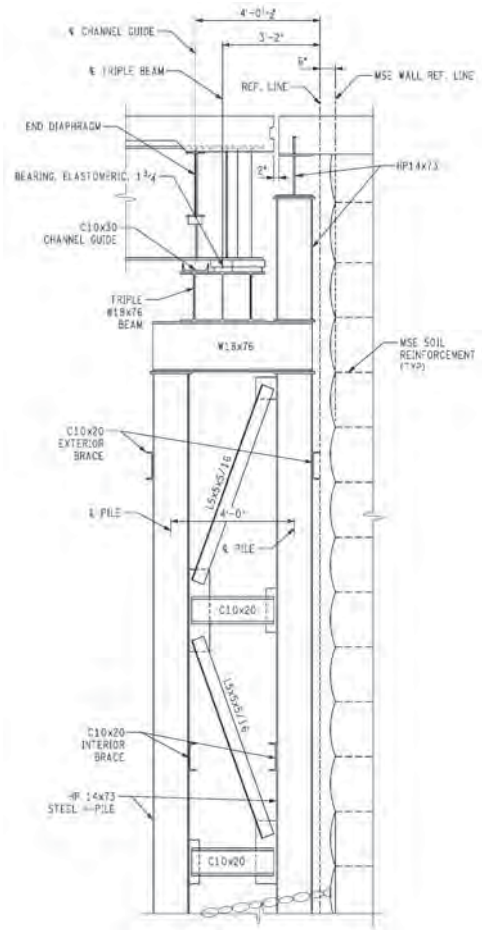

Figure 3 – Temporary Abutment Cross-Section

The frames were analyzed using STAAD.Pro V8i computer software and were designed for all applicable load cases including dead, live, temperature, and braking forces in accordance with the AASHTO LRFD Bridge Design Specifications. Temporary wire-faced MSE walls located behind the steel frames were used to retain the temporary road fills without inducing any horizontal forces on the frames. The main frame members utilized standard wide flange and pile sections. The piles were braced in each direction by standard angle and channel sections. Field welding was used to make connections instead of bolting in

order to allow for easier field fit-up. The pile location tolerance was decreased from the MDOT standard of 6” to 3”.

A lot of attention was paid to detailing connections in a way to afford the Contractor maximum tolerance. As an example, oversized plates were placed atop the piles allowing the transverse beams to be accurately placed even if the piles were misaligned. Figure 3 illustrates a cross-section through the temporary abutments.

A longitudinal triple beam was supported by the transverse beams. The triple beam (W18x76) provided a stiff member on which the temporary bearings and channel guides were placed. The triple beam spanned 8’-5” between piles in the direction of the sliding operation and was designed to limit deflections to 1/16” to minimize differential beam end deflections during the slide, thereby reducing the risk of deck cracking. The flanges of the triple beam were shop-welded together and ground smooth so that the guide channel would have a smooth level surface on which to sit. One field splice was allowed for the triple beam at the location of the drop-in span. The drop-in span was used to span the gap between the end of the temporary abutments and the end of the permanent abutments. This portion of the triple beam could not be installed until after the construction of the permanent concrete abutments. The drop-in spans were supported on one end by the steel H-piles of the temporary frame and on the other end by the permanent abutments. Initially, a removable bracket was proposed to be attached to the permanent abutments to support the drop-in span. However, due to the skew and the loading, the bracket was deemed not feasible. Instead, steel stilts were placed on top of the toe of the permanent abutment footings to support the drop-in spans.

The top of the triple beams were aligned with the beam seats of the permanent concrete abutment. The beam seats were level for the entire length of the abutments. This resulted in a continuous elevation from temporary abutment to permanent abutment. Cross-slopes of the deck were accommodated by use of variable thickness sole plates and by varying the thickness of concrete haunches.

A C10x30 channel was used at each abutment to guide the steel rollers. The channel guide channel was located in front of the elastomeric bearings of the plate girders on the temporary abutment. Like the triple beam, the channel is to be welded in the shop and ground smooth so that the rollers operate as planned. The channel is welded to the top of the triple beam and is bolted to the concrete abutment with ¾” diameter drop-in anchor studs. Any splices in the channel guides were required to be mitered to help avoid the rollers from binding during the slide operation.

APPROACH SLAB

A temporary concrete approach slab was used in conjunction with the temporary abutments. The approach slabs were tied to the temporary abutments via shear studs attached to the steel frames extending from the top of the temporary abutments. These steel frames were designed to support edge of the approach slab. This can be seen in Figure 3 on the previous page. The friction with the soil on the bottom of the approach slab also helped to stiffen the bridge in the longitudinal direction in resistance to braking forces. Since the superstructure was located in a “temporary condition”, a 2” open joint at each abutment functions as an expansion joint for the three to four months that the bridge will be open. Gravel was placed at the base of the temporary abutments to help prevent any erosion that might take place from water leaking through the open joints.

It was decided not to slide the approach slab with the superstructure due to concerns with potential interference from the permanent abutment return walls while attempting to slide the approach slab into place. Precast approach slabs were also considered for the permanent bridge, but were rejected because it was viewed as overly complicated as compared with cast-in-place construction. In the end, it was decided to use early strength cast-in-place concrete approach slabs. Threaded inserts were cast into the ends of the bridge deck to allow a connection to the longitudinal approach slab steel reinforcement. The approach slabs would act as closure pours between the superstructure and the sleeper slabs. This also allowed all of the permanent roadway pavement and sleeper slabs to be constructed prior to the slide. The expansion joint was located off the bridge between the approach slab and the approach slab, which is a typical MDOT detail. After the approach slabs achieved the necessary strength, the bridge was opened to traffic. Temporary concrete barrier was placed to allow permanent barrier construction on the approach slabs and approach guardrail installation while vehicular traffic was running the bridge.

SLIDING OPERATIONS

Both steel rollers and PTFE pads were considered. Steel rollers were selected due to prior AECOM experience with railroad bridge slides and a desire to try something different than the other two bridge slide projects that MDOT was constructing at the time. In addition, steel rollers were expected to have also have a lower breakout coefficient of friction compared to PTFE pads. PTFE pads have breakout friction values of approximately 10%

while steel roller manufacturers' literature suggests breakout friction values of approximately 5%. For design purposes, a friction value of 7% was used for this job. Pulling the superstructure would have meant extending the permanent abutment to the east in order to accommodate jacking equipment. Generally, jacks have higher capacities when extending rather than retracting. Therefore larger jacks would have been needed for a traveling jack system that pulled the superstructure into place. A stationary center hole jack could have been used with cable or post-tensioning bar, but would have prevented being able to

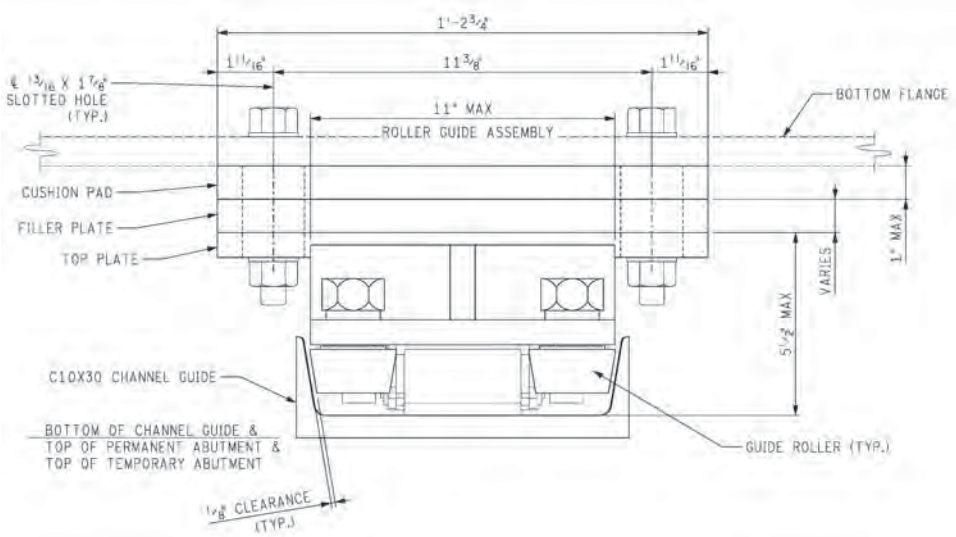

Figure 4 – End View of Steel Rollers

reverse the slide to resolve any binding of the rollers. For these reasons, it was decided to push the superstructure into place using rollers. Standard steel rollers were used except with a larger top plate fitted with slotted holes to allow for more tolerance with securing to the bottom of the beam flanges. Horizontal guide rollers were used at the fascia beams to help prevent the rollers from binding up against the channel guide flanges (see Figure 4). An elastomeric cushion pad was placed between the bottom flange of the beam and the roller assembly to help prevent damage to the proposed girder and to accommodate slight variations in flange slope.

The start of the slide operations was initiated by placing hydraulic jacks under each end diaphragm. The end diaphragms were designed for 1/16" deflection to minimize the risk of cracking the deck above. All of the jacks were linked to a synchronous lift system. The superstructure was lifted, the steel rollers were placed under the beams (sitting in the channel guides) and the superstructure was lowered on to the steel rollers. Every step of this project was analyzed to ensure proper dimensional clearances allowed for placement and removal of each component. Jacks and rollers were located in the channel guide in front of the centerline of permanent bearing so that there would be no conflict with the permanent elastomeric

bearings. This caused the permanent abutments to have a corbelled beam seat at the top of the abutment

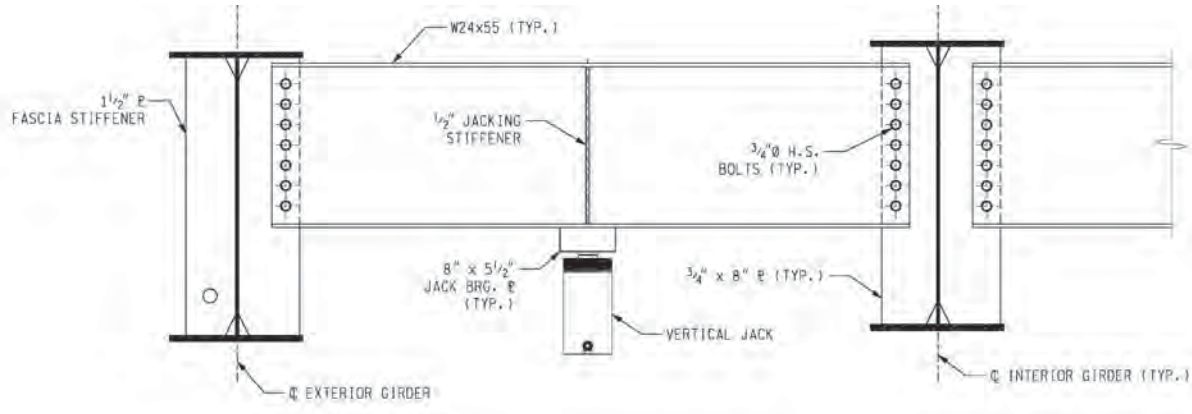

Figure 5 – End Diaphragms

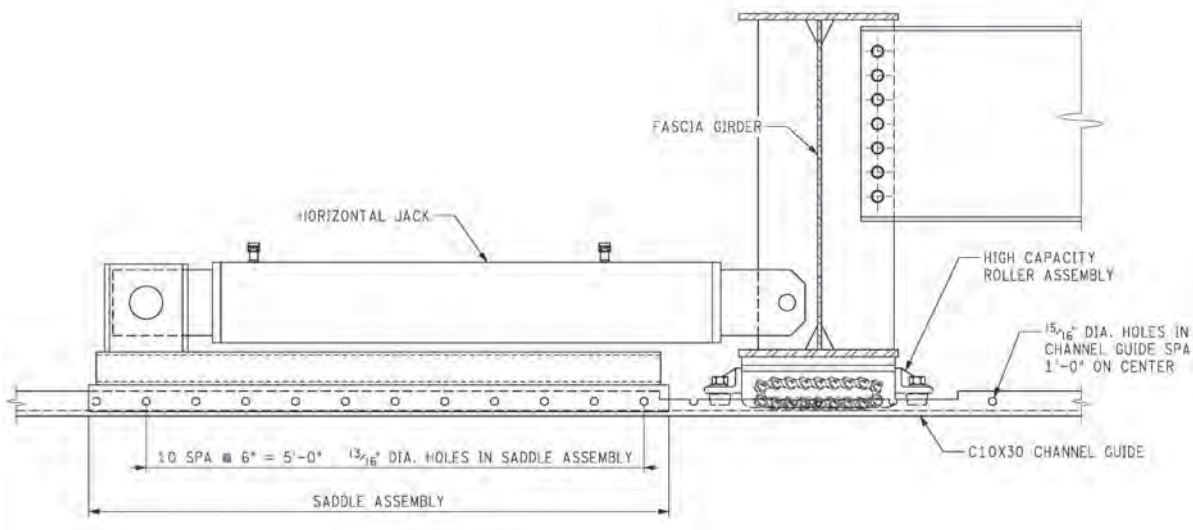

Figure 6 – Horizontal Jack Assembly

wall. See Figure 5 for a detail of the end diaphragm

Independently controlled horizontal jacks were attached to the fascia girders at each temporary abutment. The horizontal jacks were required to be double-acting to give the Contractor the ability to move the superstructure in either direction if necessary to solve any binding issues during the slide operation. The horizontal jacks were attached to a saddle assembly fabricated from steel plates, channels and a hollow structural section by means of a pin. The saddle assembly was then attached to the guide channel using 3-¾" pins. The saddle assembly incorporated holes spaced at 6". The channel guide flanges had holes spaced at 12". See Figure 6 for a detail of the horizontal jack and saddle assembly.

With the pins in place, the horizontal jacks were extended pushing the superstructure toward the permanent abutments. At the end of each stroke, the pins were removed, and the horizontal jacks were retracted to move the saddle into the next location. The pins were reinserted and the process was repeated until the superstructure was in its final alignment. At the conclusion of the slide, the hydraulic jacks were again installed under each end diaphragm. The superstructure was raised, the steel rollers were removed, and the superstructure was then lowered down onto the permanent elastomeric bearings.

MISCELLANEOUS CONSTRUCTION ITEMS

This contract was bid using the typical design-bid-build process. A detailed special provision was written for the superstructure slide which defined: tolerances, design loads, schedule, and jacking plan and contingency requirements. The Contractor was required to submit a detailed sliding plan to the Engineer for approval that described how they would control different aspects of the sliding operation. The Contractor generally followed the plans and specifications with few requests to deviate. Davis Construction Company was the low bidder on the job with a bid of $8,654,000 for the total project. The cost for the bridge over the CN/GTW railroad was $3,955,000, of which $1,944,000 was directly attributed to superstructure slide components.

HIGHWAY 406 GLENDALE AVENUE OVERPASS REHABILITATION: LATERAL SLIDE RAPID BRIDGE REPLACEMENT

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ABSTRACT

The replacement strategy for the two Highway 406 Glendale Avenue overpass structures required comprehensive evaluation and assessment of various unconventional construction methods in order to minimize impacts to traffic and the public, reduce construction duration, and ensure delivery of a cost-effective project.

Upon selection of the jack and lateral slide construction methodology as the preferred construction method, detailed design began, augmented by the identification of any inherent issues and the manner in which to approach and solve these issues well in advance of construction.

Construction, portions of which were conducted during the winter season, was completed successfully and without major consequence, a testament to the complete and thorough approach taken by the project team.

INTRODUCTION

The two Highway 406 / Glendale Avenue Overpass structures are located in St. Catharines, Ontario, and carry two lanes of Highway 406 traffic in the northbound and southbound direction, respectively, over Glendale Avenue. Both structures are superelevated and situated on a curved horizontal alignment, with 31 degree skewed support conditions and with all support lines parallel. The structures are set in a north-south direction.

The existing structures, constructed in 1962, were each comprised of three simply supported spans, with expansion joints at the abutments and piers. The abutments and piers for both structures are founded on reinforced concrete shallow spread footings. The structures had undergone considerable deterioration and were scheduled for rehabilitation of the substructure and replacement of the superstructure.

Figure 1 depicts an elevation of the existing structure, viewed from Glendale Avenue.

ventional replacement of the superstructures, using staged construction, would have resulted in unacceptable traffic impacts. As a result, a feasibility study on alternate accelerated bridge construction methods was completed and the employment of the jack and lateral slide methodology for superstructure replacement was chosen as the most favorable alternative.

The ensuing design included the rehabilitation and modification of the existing substructure, while the existing superstructure maintained Highway 406 traffic, and construction of the new superstructures on adjacent temporary supports. During separate overnight weekend closures, each existing superstructure was jacked and slid laterally onto awaiting temporary supports, the corresponding new superstructure slid into place, and traffic was reinstated shortly thereafter.

The result is two, three-span continuous superstructures with semi-integral abutments, supported on rehabilitated and modified pre-existing substructures, constructed with minimal disruption to traffic.

Figure 2 offers an aerial view of the project site.

FROM THE GROUND, UP

Maintaining, Rehabilitating and Modifying the Existing Substructures

A field investigation and structural evaluation was undertaken for the existing substructure, confirming its adequacy to support the loading of the new superstructure and current live load, as per the Canadian Highway Bridge Design Code (CHBDC). The dead load of the new superstructure was kept as close as possible to being equal to the existing dead load. Minor rehabilitation works, such as the removal of deteriorated concrete and subsequent patching, and refacing (of the north abutment only), was completed on the piers, abutments and wingwalls.

The existing substructures for both bridges were maintained, rehabilitated and modified while still supporting the existing superstructures under normal traffic loads, in an effort to minimize impacts to traffic and the public. In order to do so, new bearing pedestals had to be constructed within the constricted space between plate girder lines at the existing abutments and piers, while the existing superstructure remained in place. As can be appreciated by the cross section of both the existing and new superstructures (shown in Figure 3), little room was available to undertake such an operation, both laterally and vertically.

Meticulous survey, calculation and superposition exercises were completed to ensure that the new pedestals could be constructed without interfering with the rehabilitation and modification of the existing substructure, the existing superstructure itself and the lateral slide apparatus and operations.

In order to maintain similar abutment bearing seat elevations, while accounting for the new pedestals and bearings, the plate girders within the end spans were tapered from the pier to the abutments. Minimizing, and eliminating where possible, any substantial differences between the new structure and existing elevations helped ensure a proper fit and reduced risk of interference during the lateral slide operation. This also aided to mitigate the potential for an uneven riding surface once the new superstructures were slid into place, and abutted the existing Highway 406 roadway conditions, effectively reducing any remedial works during the defined closure period.

OUT WITH THE OLD, IN WITH THE NEW

The New Superstructures

The new superstructure cross section is comprised of five steel plate girders in comparison to the six steel plate girders forming part of the existing superstructure. This alteration was essential for rapid replacement since it allowed for the reconstruction of the substructure and greater flexibility when working around the constraints imposed by the existing superstructure and bearing pedestals. The spacing of the new plate girders was also chosen to help minimize any interference as well.

Figure 3 below shows the existing superstructure cross section in contrast with the new.

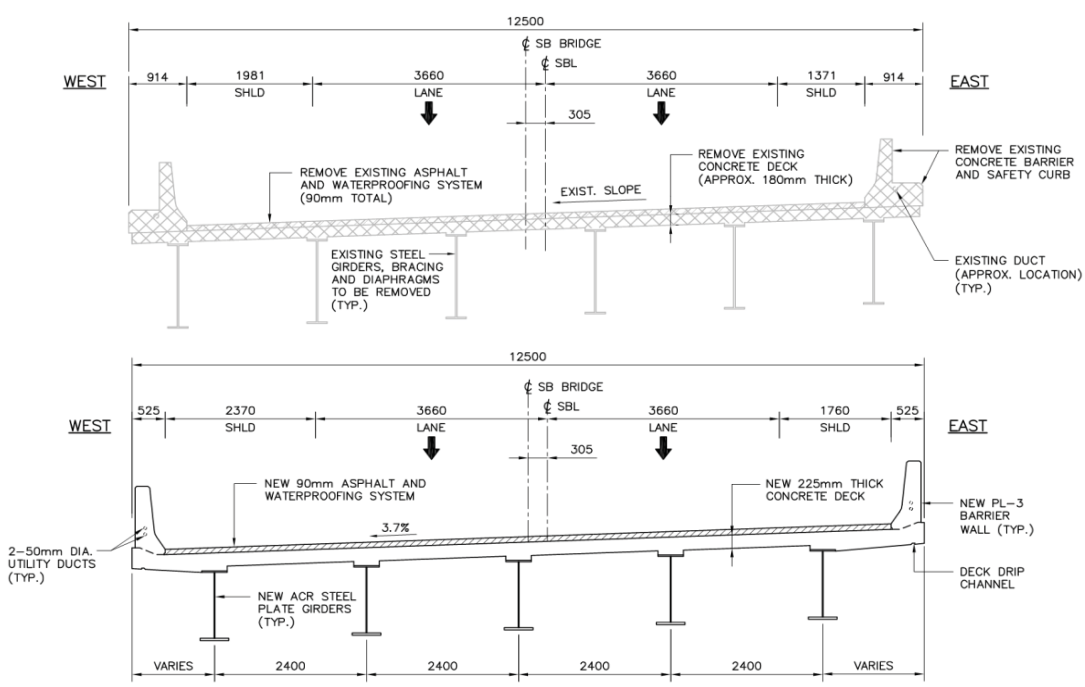

Figure 3 – Typical cross section of existing and new superstructures (southbound structure shown, northbound similar).

Since the vertical clearances on Glendale Avenue were to be maintained, the depths of the new plate girders were chosen to allow the new superstructure clearances to closely resemble the existing conditions.

Since the existing substructures for both bridges were to be maintained, the new superstructures had to be of similar span. This resulted in 12.19m end spans, and a 27.58m interior main span. Customarily, an interior span is approximately kept to within 1.33 times the end span length, but in our case, this value was an unorthodox 2.26. Since the end spans were unusually shorter than the main span, and with contribution of the skew effects, the design analysis indicated that uplift (at an ultimate limit state) would be induced in the superstructure at the abutment bearings under specific live load conditions.

Uplift could have been avoided if the plate girders within the end spans were not constrained to being tapered, which would have resulted in a deeper steel section at the abutments, and in turn, a larger, deeper, semi-integral deck end. As such, an oversized concrete diaphragm and semi-integral deck end were designed to provide ballast and to eliminate possible uplift. The deck end portion was also designed to resist the cantilever negative moment imposed on the section during the lateral slide, since it would not be supported by underlying compacted backfill, as would be the case in conventional construction.

Plate girder splices were incorporated into the main span sections to ensure that erection of the end span plate girders could occur without closing Glendale Avenue, resulting in lower risks and less disruption to traffic. Closures were only required for the erection of the main span plate girders directly over Glendale Avenue.

In order to suit the construction methods proposed, the new superstructures were designed for the anticipated construction loading stage, the lateral slide loading stage (ie. offset temporary supports,), and ultimate loading conditions.

Figure 4 below shows the erection process for the tapered end span plate girders in the photograph to the left, and the erection of the main span plate girders in the photograph to the right.

Figure 4 – Erection of end span plate girders (left) and main span plate girders over Glendale Avenue (right). (Photographs courtesy of E.S. Fox Limited)

The end spans were erected with very little disruption to traffic on Glendale Avenue and Highway 406, with access gained within the work zone between the new abutment and temporary concrete barrier. Traffic on Glendale Avenue was maintained throughout this operation. The main span plate girders were erected during a full closure of Glendale Avenue and only a partial closure of Highway 406.

For added durability, the new superstructures were made fully continuous over the piers, eliminating the previously existing expansion joints at these locations. Expansion joints were also excluded at the abutments by implementing semi-integral deck ends. By designing and constructing the new superstructures to function as jointless structures, future maintenance and rehabilitation costs are greatly reduced.

GET IN, AND GET OUT!

Accelerated Bridge Construction

This bridge site was selected as a prime candidate for accelerated bridge construction (ABC) techniques from the onset of the project. During preliminary design, MMM investigated the feasibility of several possible options, including conventional staged construction. This conventional method was not carried forward as a feasible alternative since it would reduce traffic on Highway 406 from a two lane to a one lane configuration for the entire construction duration, which would result in unacceptable traffic impacts.

Given the site's proximity to prominent businesses, it was imperative that disruption of traffic on Highway 406 and Glendale Avenue be minimized. In keeping with this necessity, MMM examined the

possible use of self-propelled modular transporters (SPMTs) as a potential candidate to help implement accelerated bridge construction.

However, the existing end spans are simply supported and so would require the use of two SPMT units to fit within the short span. In order to accommodate the dual units, the embankment in front of each abutment would have to be excavated down to roadway level to provide sufficient access. Furthermore, the abutments and remaining adjacent embankment fill would require temporary support to maintain their integrity and support the existing structure since the abutments are founded on shallow spread footings which would be undermined by the excavation. The use of SPMTs would also require the procurement of a staging yard and temporary construction site in close proximity, where the new superstructures would be built and then be transported using the SPMTs to the bridge site for installation. Utilities such as street lighting and hydro poles would be greatly impacted by this method as well, as they would have to be removed, temporarily maintained, and reinstated to facilitate transport of the new superstructure from the staging yard to its permanent location. This method was determined to be complex and uneconomical, and was eliminated from consideration.

It quickly became apparent that the most feasible and economical option for ABC would be to implement the jack and slide method. This would entail constructing a total of three sets of temporary supports, two on either side of the existing structures and one set within the median of Highway 406, between the existing structures. The new superstructures would be constructed on temporary supports immediately to the west of each existing structure, respectively. Each existing superstructure would then be jacked upwards, slid laterally, and sat down onto awaiting temporary supports, where it would be demolished, and the corresponding new superstructure slid into place.

Figure 5 below schematically shows the construction sequence employed.

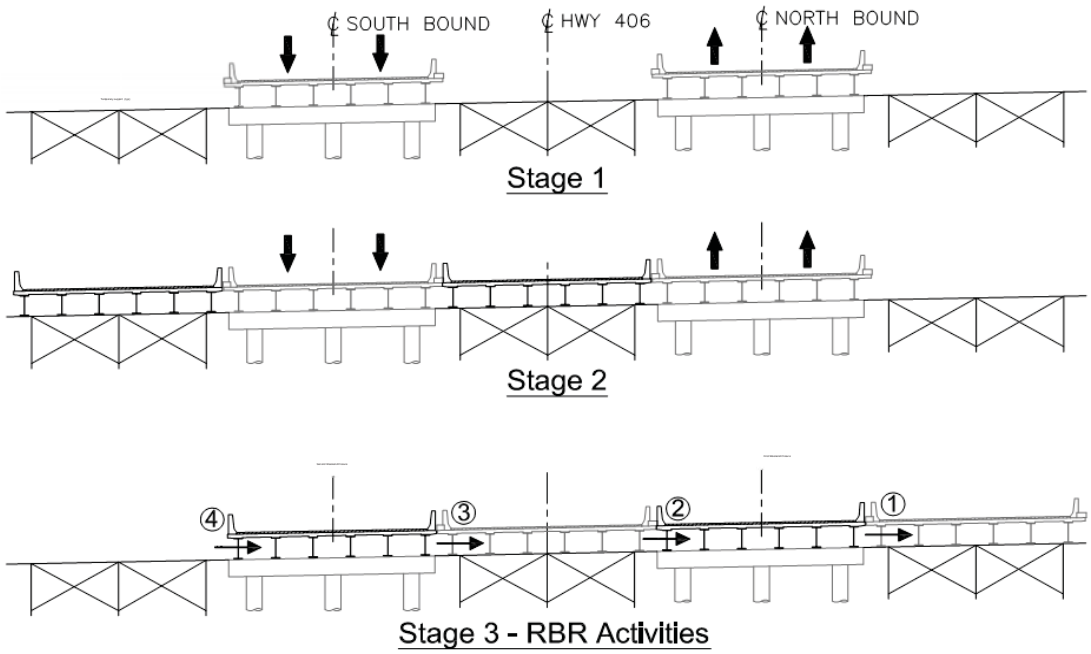

Figure 5 – General schematic of events for the lateral slide rapid bridge replacement operation.

The rapid bridge replacement works involved three sets of activities, namely: (i) Pre-Slide, (ii) Closure and (iii) Post-Slide activities. Each set is described individually herein.

Pre-Slide Activities

Pre-Slide activities involved works which would prepare each site for the lateral slide and reduce risks during the 15 hour closure period. These works included: the erection of the temporary supports and construction of the slide paths, removal of the approach slabs, sawcutting the ballast walls, the reconstruction of the substructure including new pedestals, and construction of the new superstructures on temporary supports.

Figure 6 shows a few pre-slide activities during construction.

Figure 6 – End span plate girders erected on temporary supports (left), and existing and new bearing pedestal construction (right). (Photographs courtesy of E.S. Fox Limited and Dufferin Construction Company, respectively)

Closure Activities

Lateral slides were conducted for each superstructure replacement, occurring during a 15 hour weekend closure of Highway 406 and Glendale Avenue, beginning with replacement of the northbound structure. The replacement of the southbound structure occurred on the following weekend. Each closure consisted of the jacking and sliding of the existing superstructure, repositioning of the jacking system, and jacking and sliding the new superstructure into place. Once in place, the approach to each structure was backfilled and the bridges were paved with the base course asphalt. The waterproofing system for each new superstructure was installed prior to the slide, while on the temporary supports.

The temporary support lines for the construction of the new superstructures were in-line with the existing, while the lateral slide paths were offset and parallel to the existing support lines, and positioned on either side of the piers and in front of the abutments. This configuration allowed for minimal interference between sets of supports and associated works.

Figure 7 below shows the lateral slide path setup adjacent to the temporary supports and jacking equipment.

Figure 7 – Typical lateral slide path (left) and supporting beam and vertical jacks (right). (Photographs courtesy of AECOM)

Since the existing and new superstructures were superelevated, the superstructures had to be jacked high enough to avoid the highest pedestal, slid laterally into position and lowered. During design, it was anticipated that positive 100% contact upon touch-down onto the new bearings would be unlikely, even if construction tolerances were met, and so the design specified the use of stainless steel shims between the new shoe plates and bearings.

Post-Slide Activities

Post-Slide activities posed a considerably lower risk in relation to the preceding works. These activities included the demolition of existing superstructures on temporary supports, subsequent removal of the temporary supports, removal of temporary pavement at approaches, installation of approach slabs and paving the final top course of asphalt.

Originally, the Contract allowed for the construction of the approach slabs during four separate full closure periods of 11 hours each using rapid set concrete. Anticipating that the approach slabs could very well be constructed in a different manner, dependent upon the Contractor's abilities, the Contract also included a provision which allowed a Contractor proposal for the use of prefabricated (precast) approach slabs. The requirements of which were included in the Contract Specifications.

Upon submission of a Contractor proposal, and after its careful consideration and evaluation, it was determined that the approach slabs, consisting of an interior three panel precast portion and two exterior cast-in-place sections, at each bridge end, could be successfully installed in two successive weekend closures, of 16 hour durations each. Similar to the superstructure, the northbound structure's approach

slabs were completed during the first closure weekend, followed by the southbound structure the following weekend. Closure strip joint pours, using a high early strength concrete material, were introduced longitudinally between the panels and transversely at the semi-integral deck end in order to provide continuity with the superstructure.

The works during each closure consisted of closing the highway and redirecting traffic onto the adjacent ramps, removal/excavation of the temporary asphalt and fill material at the abutments, placement and compaction of granular fill material, installation of the interior panels and pouring the closure strip joints. After the concrete had reached the minimum specified 20MPa compressive strength (approximately within a 4 hour period), the approach slabs and approaches were paved with two lifts of asphalt. The final lift of asphalt was also paved on both structures during this closure.

The exterior portions of each approach slab, which were designed to be integral with the barrier walls and be supported by, and slide overtop of, the underlying wingwall, were constructed as cast-in-place portions during regular daytime work hours. Construction progressed on these exterior portions without any lane closures on Highway 406, behind a temporary concrete barrier, delineating traffic on Highway 406 from the work zones on either side.

Figure 8 shows the construction of the precast approach slab panels on site and a view of a typical closure strip joint.

Figure 8 – Construction of approach slab panels (in a staging yard) (left) and view of transverse closure strip at the semi-integral deck end. (Photographs courtesy of AECOM)

COMPLETION OF THE PROJECT

The two superstructure lateral slides, both existing and new, were completed in January 2015. Work for the approach slabs, including paving operations, was completed in June of this year, and traffic fully restored in early July.

Currently, both new structures are fully operational and currently carrying normal Highway 406 traffic.

ACKNOWLEDGEMENTS

Sincere thanks to the Ministry of Transportation for the opportunity to work on this challenging project.

The successful completion of this complex project was made possible by the following team:

- Dufferin Construction Company, General Contractor;
- Burnco Manufacturing Inc., Steel Fabricator/Detailer;
- E.S. Fox Limited, Steel Erector;
- AECOM, Contract Administrator;
- Mammoet, Lateral Slide;
- Brown & Co., Erection Engineering;
- WP Engineering, Design of Temporary Works; and
- Domson Engineering, Engineered Lift Plans.

LATERAL SLIDE OF HISTORIC BRIDGE IN WASHINGTON STATE

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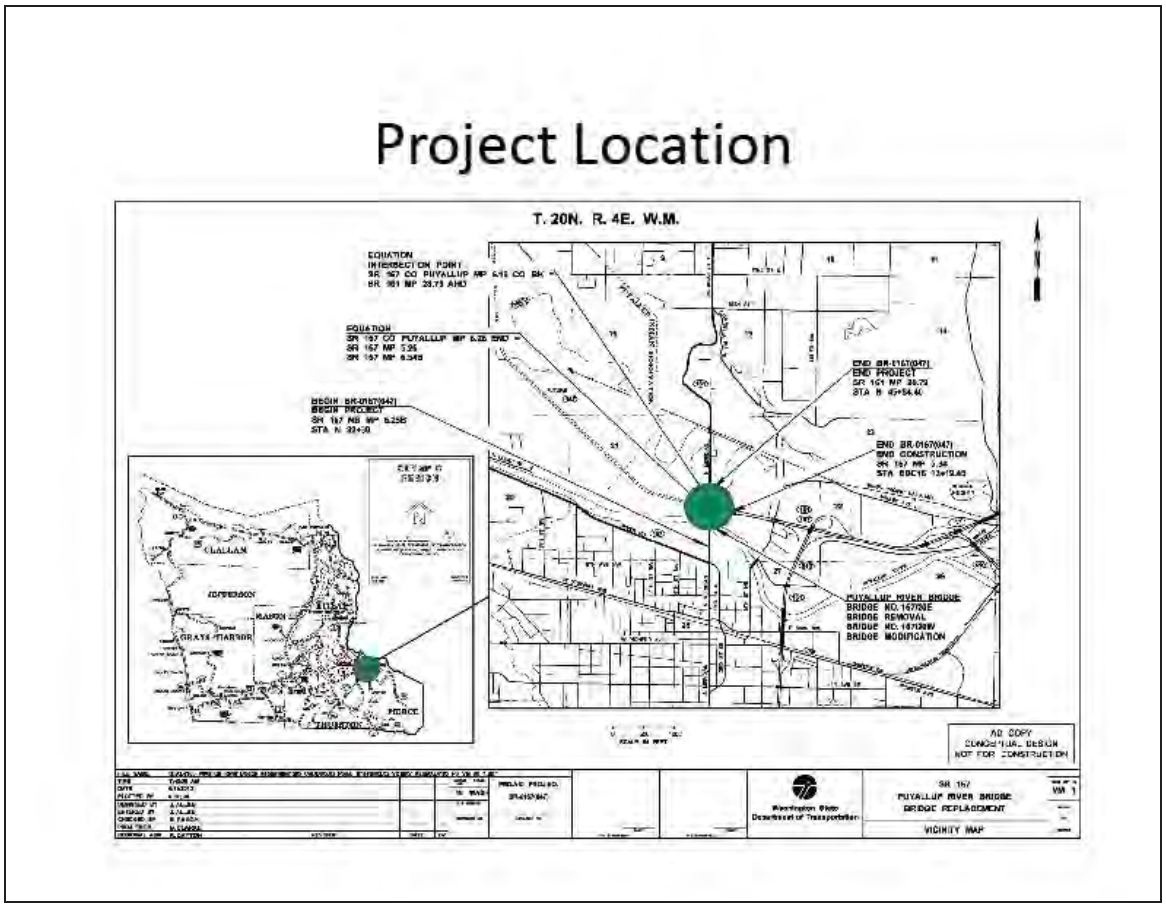
INTRODUCTION

To shorten construction time and minimize construction costs, an existing truss was moved to become a detour bridge. Moving of the bridge allowed for use of the existing roadway alignment and infrastructure for the permanent construction. Preserving the existing alignment eliminated the need for retaining walls, barrier and sidewalk modifications to the existing SB bridge, new signals and major utility relocation. Elimination of new roadway elements greatly reduced construction time and costs. The presentation will demonstrate how the 371' long, 1.5M pound truss was moved and lessons learned along the way.

PROJECT LOCATION

The project is located in the City of Puyallup, Washington along State Route 167, over the Puyallup River.



PROJECT DESCRIPTION

This Washington State Department of Transportation (WSDOT) project utilized the Design Build method of construction to replace a functionally obsolete and structurally deficient 1925 Warren truss. A new two-lane bridge with 12’ lanes, a 4’ left shoulder, a 2’ right shoulder and an 8’ sidewalk was constructed within the original alignment. The Warren truss was listed on the National Historic register and it is hoped that the truss will be relocated and repurposed to become part of a multi-use trail in the future. At the completion of the job the truss was moved in one piece and is being stored on an adjacent site. (https://www.youtube.com/watch?v=bmzmtlwbDA4&feature=youtu.be) The design build project was won by Atkinson Construction and Jacobs was the primary design consultant.

The Request for Proposal included preliminary plans for a new alignment with a four span precast, prestressed concrete girder bridge with piers in the river. Atkinson proposed utilizing the existing alignment and replacing the bridge with a three span steel plate girder bridge with no piers in the river. In order to incorporate the existing roadway and infrastructure into the new project a detour was necessary. The river crossing for the detour utilized the existing 371’ truss, which had to be moved laterally 56.6’ and vertically 2’ to make room for new bridge construction. The truss move was completed in July 2014

using the slide method. The presentation will demonstrate the equipment necessary for sliding a 1.5 million pound structure and the planning required to complete such a move. One weekend was allowed for the move of the truss and shifting of traffic onto the detour alignment.

The presentation covers:

- Project overview
- Detour Plans
- Truss move Plans
- Truss moving equipment
- Truss moving videos and timelapse video
- Lessons learned

G-5: ABC SUBSTRUCTURES SOLUTIONS

POST-TENSIONED PRECAST PIER CAPS FOR THE HONOLULU RAPID TRANSIT PROJECT

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ABSTRACT

The Honolulu Authority for Rapid Transit (HART) is a semi-autonomous public transit authority created in 2011 and charged with building a light rail rapid transit system on the Island of Oahu in Hawaii. The goal of the system is to alleviate severe traffic congestion on the H1 highway by constructing an entire light rail mass transit system from scratch. The project is twenty miles long starts in East Kapolei goes to Honolulu International Airport and ends at the Ala Moana Center in downtown Honolulu. Almost all of the route is on an elevated viaduct. There are twenty one stations each will have high level platforms, state of the art security features and be fully compliant with the Americans with Disabilities Act. It is estimated that by 2030 there will be 119,300 daily passenger trips. The system will operate 4 AM to Midnight daily with trains every 5 minutes during rush hours and special events, and 10 minutes during non-rush hours. Estimated project cost is six billion dollars and is being delivered by a combination of Design-Build alternative delivery for the light rail system, and Design Bid Build for the stations. Construction began in 2011 and the rail service from East Kapolei to Aloha Stadium (first ten miles) scheduled to start in 2018 and be fully open in 2019. To the scheduled opening and complete all the testing and certifications requires an aggressive accelerated construction schedule with many challenges and restrictions.

My presentation will discuss the light rail viaduct piers located at the West Loch and Waipahu Transit Stations. The presentation outline is as follows:

1. Introduction
2. Design and detailing of the piers
3. Planned construction staging of the piers
4. Coordination with station designers
5. Actual construction of piers
6. Summary

1. INTRODUCTION

The West Loch and Waipahu Transit Stations are located in the first segment of the system between East Kapolei and Aloha Stadium, and will be in service starting in 2018.

The pier layout for both stations is the same. There are six piers; two approach piers, two hammerhead end piers, and two middle piers closely spaced and connected with a tie beam that will support a pedestrian bridge.

The piers are situated in the median of an active roadway that will remain open throughout construction.

West Loch Station

This station is located over Farrington Highway near the intersection of Leo'ole Street. Station piers are numbered 156 to 161. The piers support spans 155 to 161 (145', 102', 85', 30', 90', 145' and 145' lengths). The station is located in Seismic Design Category B as defined by the 2007 AASHTO Guide Specifications for LRFD Seismic Bridge Design.

Waipahu Station

This station is located over Farrington Highway near the intersection of Mokouola Street. Station piers are numbered 211 to 216. The piers support spans 210 to 216 (125', 106', 85', 30', 90', 145' and 145' lengths). The station is located in Seismic Design Category C as defined by the 2007 AASHTO Guide Specifications for LRFD Seismic Bridge Design.

2. DESIGN AND DETAILING OF PIERS

Design Criteria and Train Loads

Honolulu High-Capacity Transit Corridor Project Compendium of Design Criteria

Design Criteria for Bridges and Structures, HDOT

AASHTO LRFD Bridge Design Specifications, 4th Edition, 2009

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2007

AREMA Manual for Railway Engineering, 2009

CEB-FIP Model Code for Concrete Structures, 1990, Chapter 2: Material Properties

WSDOT Bridge Design Manual, 2010 and WSDOT Report WA-RD417.1 for Non-Contact Lap Splices

Live Loads

Design live load consists of Light Metro Vehicle (LMV) in a four vehicle train (240' long train) arrangement. Each LMV weighs 117 kips on four axles.

The alternative live load is a maintenance train consisting of a single HP car 58' long. The HP car weighs 120 kips on four axles.

Impact factors are 33% vertical (IMV) and 10% horizontal (IMH) of total live load.

Accelerating Pier Design (Accelerated Bridge Design – ABD)

The West Loch and Waipahu are nearly identical in span arrangement and pier height. However, their seismic design categories are different and so are their soil profiles. Waipahu has slightly taller piers and is in the more restrictive seismic zone. To complete the design for both stations within the very short design schedule the decision was made to design and detail the Waipahu Station piers first and re-use the design and details of as many components as possible for the West Loch Station piers. Justification of design and detailing re-use would be technical memo showing the results of a loading comparison analysis.

Drilled Shafts and footing

The drilled shafts and footing designs for each station the only components that are unique to each station due to differing soil conditions and seismic categories.

Column and Capital

The columns and capitals were designed for Waipahu and re-used for West Loch. Seismic Design Category C requires the development of plastic hinging in the columns and a check of over-strength to ensure hinging does not occur anywhere else. West Loch piers do not require these conditions (SDC B), but the costs of the additional material for meeting these conditions is insignificant compared to the cost of the engineering to design SDC B columns.

Pier Caps

The pier caps consist of three precast concrete sections (A, B and C) post-tensioned together in two stages. Sections A and C are identical and cantilever off section B. Section B is centered over the column. The concept is to install two sections (A and B) and post-tension them together while traffic is stopped on the roadway below Section A. Section C is then installed and post-tensioned while traffic is stopped below Section C.

Quality Assurance and Quality Control

Issue for Construction

3. PLANNED CONSTRUCTION STAGING OF PIERS

Foundation Construction

Column Construction

Pier Cap Staging

4. COORDINATION WITH STATION DESIGNERS

Assumed temporary construction loading for station construction

Consistency of details between design plans

Design Criteria and Train Loads

5. ACTUAL CONSTRUCTION OF PIERS

A Design-Build alternative delivery project means that the design engineers work for the construction contractor and not the owner. It means that the engineer is working on behalf and in the best interest of the Contractor and at the Contractor's direction; it is an important distinction for engineers to understand.

Building a multi-billion dollar 20 mile long light rail system is incredibly complex to the point that even the best laid assumptions that won the project and best laid designs and detail will undergo significant change once construction begins. Change Management is a significant work task in project of this magnitude and complexity.

Field Design Change of Pier Caps

After the West Loch Station pier design, plans and specification were issued for construction, and foundation construction had started, the Contractor directed us to re-design the pier caps to be cast in place instead of precast citing costs that had changed from the initial bid that had won the project to now favoring CIP over Precasting. The directive tasked us with re-designing on an extremely short schedule to stay ahead of construction that was already underway.

Cost comparison between Precast Caps and Cast-In-Place Caps

6. SUMMARY

Lessons Learned

Alternative Delivery Projects demand Accelerated Bridge Design (ABD) to provide quick responses to change.

The cost of ABC techniques may change enough in time as to make them less favorable after construction starts.

ABC in Alternative Delivery

ABC in Mass-Transit Projects

Conclusion

EXPERIMENTAL STUDY ON PRECAST SEGMENTAL BRIDGE COLUMNS WITH SEMI-RIGID CONNECTIONS

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Kuo-Chun Chang, Prof., National Taiwan University, Taiwan
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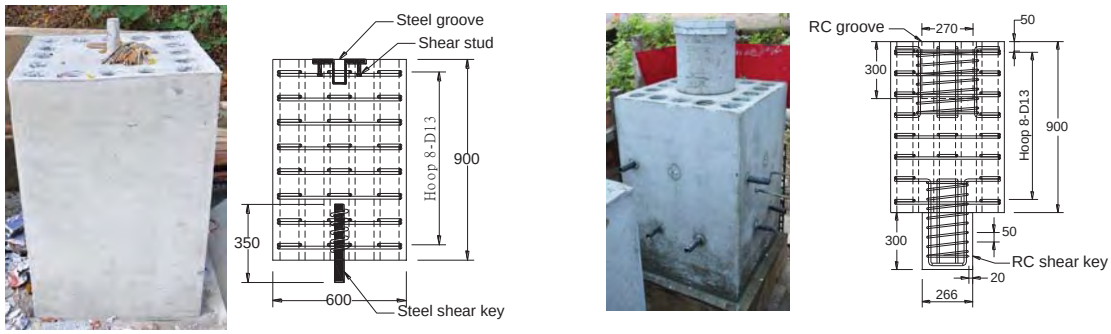
ABSTRACT

To prevent the large axial prestressing forces that are usually necessary for a posttensioned precast segmental column for providing sufficient friction forces between each segment, a semi-rigid connection for the precast segmental bridge pier system was proposed. The proposed connection between segments is a hybrid connection contains bonded bar reinforcement that is spliced by bar couplers and shear keys to provide shear resistance between neighboring segments. Unbounded tendons with a small prestressing force could be included too to provide re-centering forces. Two types of connection between each prefabricated element, i.e., steel dowel shear key and RC shear key, were proposed and tested at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. From the experimental results and construction practices of the developed system, the high seismic resistance and the satisfactory constructability of the proposed pier were confirmed.

INTRODUCTION

Since the use of precast segmental technology allows bridge engineers to reduce traffic disruptions, to minimize environmental impact and to increase the speed of construction, it has been widely applied on bridge construction worldwide recently. Nowadays, the most commonly used precast segmental technology for substructures is the post-tensioned precast segmental column, such as the work done by Hewes and Priestley (1), Chou and Chen (2), Kima *et al* (3) and Ou *et al* (4), among the others. For most of these designs, a high value of post-tensioning was typically adopted for the precast segments to provide sufficient axial force that imposes friction between the neighboring segments. The prestressing can provide resistance against shear stress caused by an external force and also provide re-centering force. However, as the bridge column bears large axial forces even no external force applied thereto, it may cause adverse effects on the ductility of the bridge column and may result in excessive stress on the precast segments. In addition, providing reliable connections to ensure ductile performance is essential for

the designs in earthquake-prone areas (5). To address above issues and to develop a system for potential use in seismic regions, a semi-rigid connection for the precast segmental bridge pier system was proposed in this study by extracting the concept of human spinal column. The proposed connection between segments is a hybrid connection contains bonded bar reinforcement that is spliced by bar couplers and shear keys to provide shear resistance between neighboring segments. Unbounded tendons with a small prestressing force could also be included to provide re-centering forces. For the proposed system, the bearing elements of bonded reinforcing bars can provide strength and energy dissipation capacity, and the prestressing elements can provide re-centering force upon the column deformation. Only small amount of prestress force is required for the column owing to the provision of the shear keys for the precast segments against shear stress induced by an external force. Compared to the conventional methodology, the present design can resolve the issue of large axial pressure loading on the column caused by excessively prestressing. In particular, the precast segments of the present invention can be stacked by mortise-and-tenon joints so as to prevent lateral displacement and address the issue of high prestress force required for the conventional art. In this study, two types of connection between two adjoining prefabricated elements, i.e., steel dowel shear key and RC shear key, were proposed and tested at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. From the experimental results and construction practices of the developed system, the high seismic resistance and the satisfactory constructability of the proposed pier was confirmed.



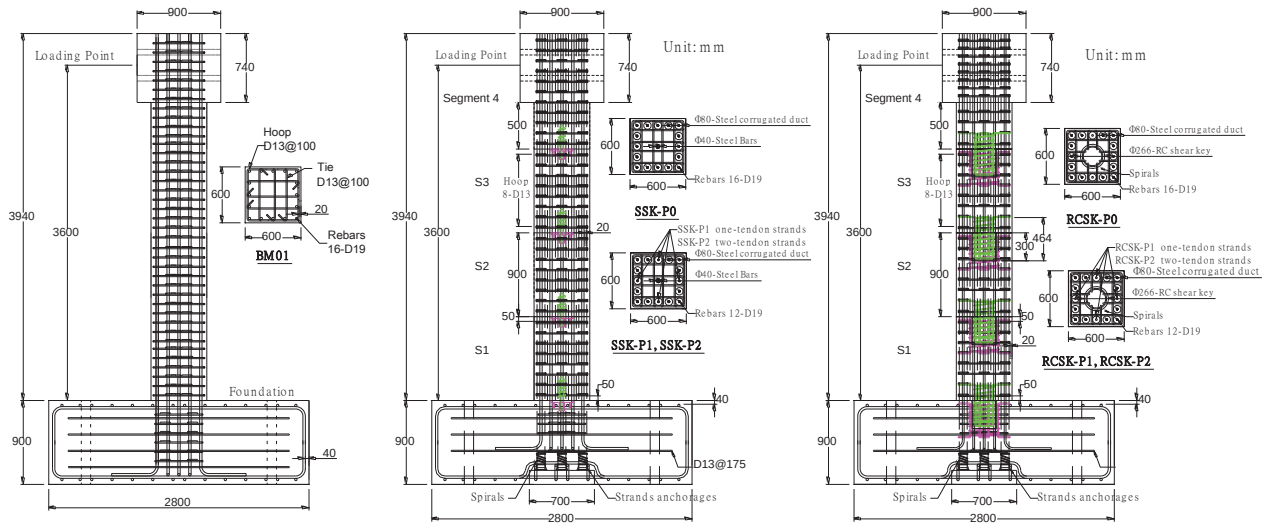
(a) (b)

Figure 1 Design details and photo of shear keys: (a) steel dowel shear key; (b) RC shear key

SPECIMEN DESIGN

In this study, two types of connection between each prefabricated element were proposed. One was steel dowel shear key and the other was RC shear key as given in Fig.1. The steel dowel shear key was made of JIS G4051 S45C steel bar with a diameter of 40 mm. The RC shear key was made of reinforced concrete with a diameter of 266 mm and Integrated with the segment. Both connections were designed based on

the concept of mortise-and-tenon joints. Each connecting set includes a shear key and a joint hole, wherein the shear key protrudes from one surface to serve as a tenon tongue and the second surface has a joint hole to serve as a mortise hole. Thus, the precast segments of the present design can be stacked by mortise-and-tenon joints so as to prevent lateral displacement and address the issue of high prestress force required for prestressed tendons.



(a) (b) (c)

Figure 2 Design details of specimen: (a) specimen BM01; (b) specimens SSK-P0,P1,P2 (c) specimens RCSK-P0,P1,P2

To investigate the seismic performance of both types of connection, three precast segmental column specimens with different prestressing forces for each type were constructed and tested at the National Center for Research on Earthquake Engineering (NCREE). For the columns adopting steel dowel shear keys, the specimens were denoted by SSK-P0, SSK-P1 and SSK-P2. For the columns with RC shear key, the specimens were denoted by RCSK-P0, RCSK-P1 and RCSK-P2, where P0 represents zero prestressing force, P1 and P2 represent a prestressing force of 0.02 fc'Ag and 0.05 fc'Ag, respectively. For comparison, a conventional cast-in-place monolithic column specimen without connections (specimen BM01) was also constructed and tested. Thus, a total of 7 specimens, as given in Fig. 2, were constructed and tested at NCREE. As can be observed in Fig. 2, all these specimens were 3.6 m in effective height with a cross section of 60cm × 60cm. The equivalent monolithic specimen with the conventional details was reinforced with 16-D19 SD420 rebars (steel ratio = 1.27%) and transversely reinforced with D13 perimeter hoops and internal stirrups spaced 10 cm. Both P0 specimens were also reinforced with 16-D19 SD420 rebars, while P1 and P2 specimens were reinforced with 12-D19 SD420 rebars and 4 tendons. Each tendon for specimens P1 and P2 comprised of one and two seven-wire strands

made of steel equivalent to ASTM A416 Grade 270, respectively. Each strand had a nominal diameter of 15.2 mm. Other detailed design parameters for all these specimens are given in Table. 1. Each prefabricated specimen was 4.84 m in total height and consisted of a foundation, three precast column segments and a precast cap beam. The height of each column segment was 90 cm and each segment was precast with 16 corrugated steel dusts inside to allow the reinforcing bars and tendons to pass through, and integrated with a shear key and a joint hole. The vertical reinforcing bars were divided into two parts, and connected to each other by mechanical couplers in two different planes. The corrugated steel dusts with the reinforcing bars passing through were pressure grouted after all the segments were stacked, but that with the tendons were not grouted to let the tendons remain unbonded.

Table 1 Properties of test specimens

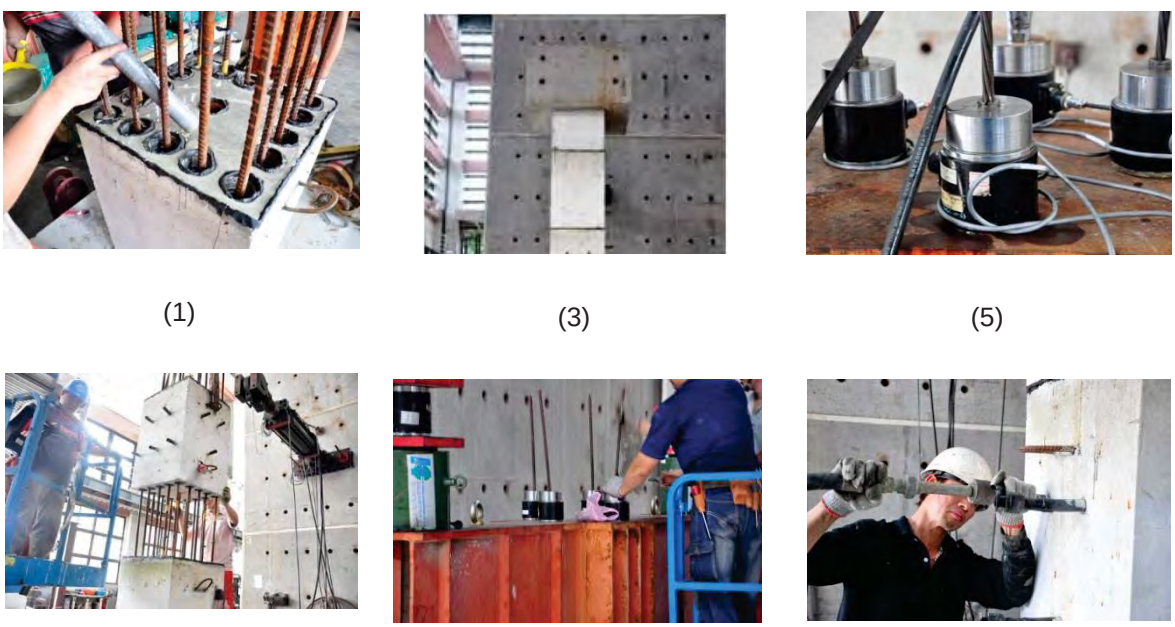
Specimen	Concrete strength MPa	Yield strength for the main bars MPa	Longitudinal reinforcement -	Tendon	Column height mm	Axial load ratio -	Prestressing force ratio -
BM01	36.6	466.5	16-D19		3600	$0.1f_c'A_g$	-
SSK-P0	36.6	466.5	16-D19	-	3600	$0.1f_c'A_g$	-
SSK-P1	36.6	466.5	12-D19	4-15.2mm	3600	$0.1f_c'A_g$	$0.02f_c'A_g$
SSK-P2	36.6	466.5	12-D19	8-15.2mm	3600	$0.1f_c'A_g$	$0.05f_c'A_g$
RCSK-P0	42.4	472.7	16-D19	-	3600	$0.1f_c'A_g$	-
RCSK-P1	42.4	472.7	12-D19	4-15.2mm	3600	$0.1f_c'A_g$	$0.02f_c'A_g$
RCSK-P2	42.4	472.7	12-D19	8-15.2mm	3600	$0.1f_c'A_g$	$0.05f_c'A_g$

CONSTRUCTION PROCESS

For the precast segmental specimens, all segments were prefabricated in advance and the specimens were erected at NCREE. After the foundation segment was placed in position, each column specimen was constructed by stacking precast segments on top of each, and the reinforcing bars, which were precast in foundation segment and protruded from the top surfaces of the foundation, were passed through the corrugated ducts precast within the column segments. Some of the erected photos is given in Fig. 3 and the process is briefly introduced as follows: (1) the top surface of the bottom segment was leveled by pouring non-shrink grout on its top; (2) The top segment was placed onto the bottom segment and the main bars were passed through the corrugated ducts which were precast within the segment; (3) All the segments were placed; (4) For the specimens with tendons, the tendons were anchored at the top of the cap beam and the bottom recess of the foundation. (5) The tendons were post-tensioned from the top of the cap beam; (6) The corrugated ducts with the reinforcing bars passing through were pressure grouted, while the corrugated ducts with the tendons passing through were not grouted.

EXPERIMENTAL PROGRAM

In order to investigate the seismic performance of the proposed precast segmental columns with semi-rigid connections, cyclic loading test were conducted on these 7 specimens at NCREE. Fig. 4 illustrates the test setup. Four high tensile strength tie-down rods were placed through the footing and anchored into the strong floor of the laboratory to simulate the fixed-base condition of the foundation. During the test, an axial load of 1260 kN was applied to the test column through a tap beam using two vertical high tensile strength rods. The vertical loading was kept constant throughout the test to simulate the tributary dead load of the deck, which was around 0.1 $f_c'A_g$. In which, A_g is the gross cross-sectional area of the column and f_c' is design concrete compressive strength. In addition, one horizontal actuator was used to apply the lateral force to the column's top to simulate the seismic loading. The location of the application force was 3.6 m up from the top of the footing. Displacement-controlled cyclic loading test was performed on these specimens. Fig. 5 shows the displacement loading protocol for the test, where the excited drift ratios include 0.25%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0%, 5.0%, 6.0%, 7.0%, 8% and 9%. The prescribed displacements were applied on the column three cycles for each drift ratio which is equal to or lower than 3%. For the drift ratio other than these values, the corresponding lateral displacement was applied on the column top for two cycles.



(1) (3) (5)

(2) (4) (6)

Figure 3 Construction Photos

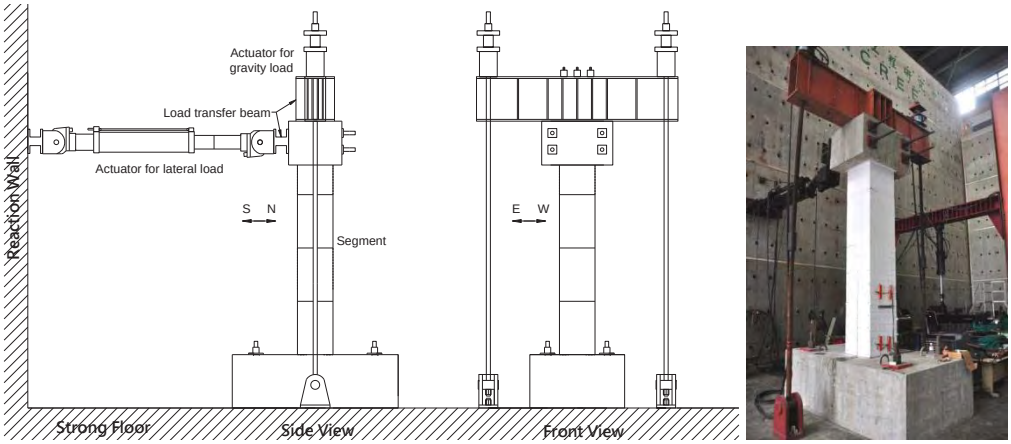


Figure 4 Experimental setup

TEST RESULTS

Fig. 6 shows the load-displacement hysteretic curve for each specimen. These figures show that all specimens exhibited stable ductility behavior under cyclic loading up to 6% drift. The failure mode for all of these seven specimens was flexural failure and the strength degradation was due to fracture of thr vertical reinforcing bars. For the conventional cast-in-place monolithic column specimen BM01, the loss of strength occurred at the second cycle of 6% drift, immediately after the fracture of two vertical reinforcing bars. More vertical reinforcing fractured at the first cycle of 7% drift. Thus the test terminated after the excitation for 7% drift. For specimens SSK-P1 and SSK-P2, the test stopped at 8 % drift and the tests for all the other specimens were terminated at 9% drift after the loss in strength exceeded 20%. By comparing the hysteretic force displacement response between the precast segmental columns without pretressing tendon (specimens SSK-P0 and RCSK-P0) and the cast-in-place monolithic column specimen BM01, one can find that the precast segmental columns without prestressing tendon can not only emulate the hysteretic behavior of the conventional cast-in-place monolithic column specimen BM01, they also had a better ductility capacity. In addition, by comparing the precast segmental columns with different prestressing forces, one can note that with an increase in the prestressing forces, the residual displacements have a tendency to be reduced, especially for the precast columns with steel dowel shear keys. However, the increase in the post-tensioned prestressing force also accompanied by a slight reduction in ductility capacity of the column due to the increase in the axial forces.

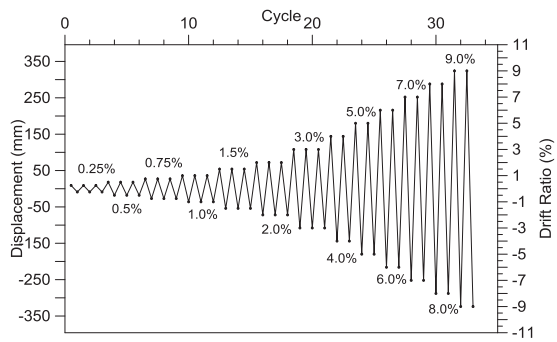
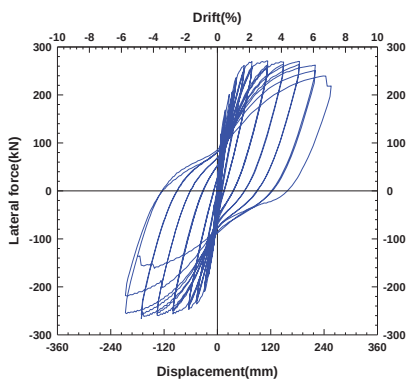


Figure 5 Loading protocol

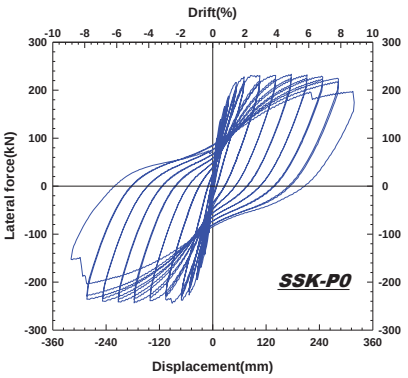
Fig. 7 shows the residual drift of each specimen under different drift ratios, where parts (a) and (b) show the comparison results of the conventional monolithic column specimen BM01 with the specimens with steel dowel shear keys (SSK specimen) and RC shear keys (RCSK specimens), respectively. The residual drift was defined as the average of absolute values of lateral drifts at zero force for each level of cyclic loading. By comparing the benchmark specimen BM01 with the specimens without prestressing forces, the residual drift for specimen SSK-P0 was slightly smaller than that of the benchmark specimen BM01, and the residual drift for specimen RCSK-P0 was close to that of the benchmark specimen BM01. This observation indicates that the response behavior for the precast segmental specimens without prestressing forces was similar to that of the conventional cast-in-place specimen, even though the ductility for the precast segmental specimens was better than that of the benchmark specimen. On the other hand, for the specimens with prestressing forces, the residual drift for the specimen SSK-P1 was far lower than that of the benchmark specimen, especially as the displacement drift reached 4%, and so did specimen SSK-P2. For the specimens with RC shear keys, i.e., specimen RCSK-P1 and RCSK-P2, similar phenomenon can be observed. However, the reduction in residual drift due to the prestressing forces was not as significant as that can be observed for the counterpart specimens with steel dowel shear keys. This results probably due to the reason that the size of RC shear key was much larger than that of the steel dowel shear key and imposed more constraint between adjoining segment to limit the rocking mechanism between them, as a result the specimens exhibited less joint opening between segments and presented more hysteretic behavior.

Fig. 8 and 9 show the vertical distribution of the curvature in the potential plastic hinge region for each test column after the excitation of the first cycle for each drift ratio. The curvature was obtained by taking the difference between the readings of two adjacent tiltmeters divided by the distance between them. In these figures, the influence of joint opening on the curvature variation was not shown. Therefore, the curvature given in Figs. 8 and 9 can represent the extent of damage of the specimens. It is obvious that the curvature of the conventional monolithic column specimen given in Fig. 8 was much larger than of the

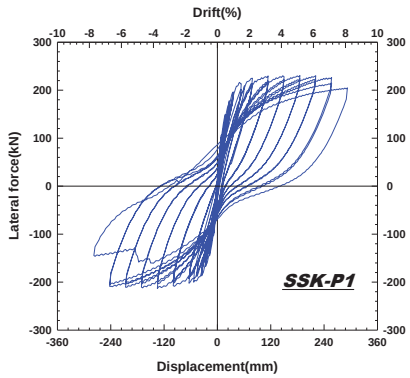
precast segmental columns shown in Fig. 9 and was evenly distributed along the location with a distance of 0~600 mm from the top of the foundation. This comparison illustrates that the precast segmental columns in this report suffered much less damage than the conventional monolithic column. Other phenomenon can be observed in Fig. 9 is that the curvature for the specimens with RC shear keys was higher than that for the specimens integrated with steel dowel shear keys. This observation implies that the damage level for the specimens with RC shear keys was more serious than that with steel dowel shear keys. This is due to the reason that the constraint against rocking of the segment imposed by the steel shear keys was smaller than the by the RC shear keys, and therefore the uplift of bottommost segment of SSK specimens was larger than that of RCSK specimens, as is visible in the plots shown in Fig. 10. Fig. 10 shows the joint opening histories of all the specimens. It can be seen that joint between foundation and the bottommost segment S1 of SSK specimens experienced larger opening than the corresponding RCSK specimens. The geometric nonlinear behavior of joint opening can help to absorb some earthquake energy and mitigate the damage of the column itself.



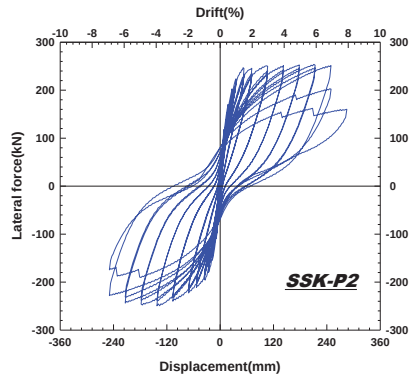
(a) BM01



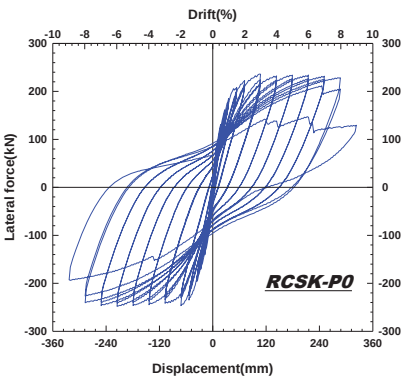
(b) SSK-P0



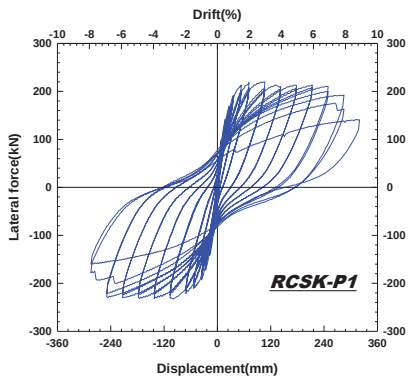
(c) SSK-P1



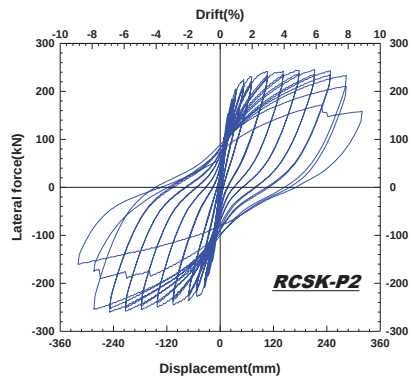
(d) SSK-P2



(e) RCSK-P0

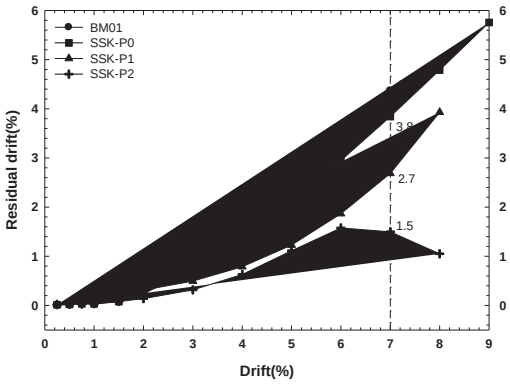


(f) RCSK-P1

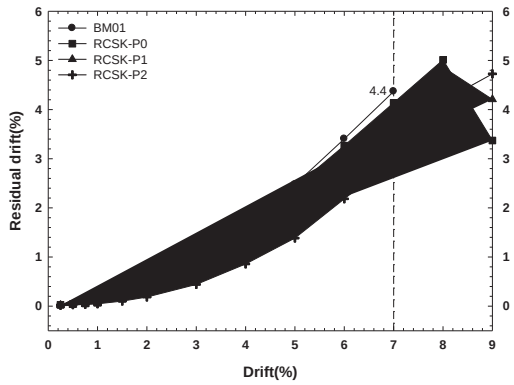


(g) RCSK-P2

Figure 6 load-displacement hysteretic curves



(a) BM01 & SSK



(b) BM01 & RCSK

Figure 7 Residual drifts

Similar conclusion can also be drawn from the failure photos of each specimen taken after the tests. The failure photos for the conventional monolithic column specimen and the precast segmental column specimens are given in Fig. 11 and 12, respectively. For the conventional column BM01, Fig. 11 shows its failure photos taken after the excitation of drift ratio 7%. For the precast segmental columns, Fig. 12 shows the photos taken after loading up to 9% drift ratio for specimens SSK-P0, and RCSK-P0/P1/P2 and photos taken after loading up to 8% drift ratio for specimens SSK-P1 and P2. For specimen BM01, most of the cracks were focused on the plastic hinge zone. At drift ratio of 3%, compression crushing of cover concrete started to occur at the corner and the flexural crack became significant. Flexural cracks first appeared on the north and south faces of the columns, and gradually propagated in the form of flexural-shear cracks as appeared on the east and west faces as the drift increased. At drift ratio of 5%, large chunks of concrete started to spall and the transverse reinforcement was partially exposed. At the second cycle for drift of 6%, both the transverse and vertical rebars were exposed and the vertical reinforcing bars started to fracture. Moreover, at the end of the test, the bulging of the rectilinear hoops caused by the large lateral dilation of the concrete was serious and resulted in the failure of cross ties at the 90 or 135-degree bends. Consequently, some of the hook of cross ties were open up and followed by buckling of the nearby longitudinal bars

For the precast specimens SSK-P0、SSK-P1、SSK-P2, which have the connections of steel dowel keys, the concrete at the corner also started to crush at the drift ratio of 3%, at the same time, the flexural cracks can also be observed. However, the failure of concrete was much less in the segmental columns than the standard column and the observed cracks were not as significant as can be observed from the monolithic column BM01. At the drift ratio of 5%, chunks of concrete started to spall, but the spalled concrete was only limited to the cover concrete. Thus, the strength of the specimen was not degraded at this stage. The first loss of strength occurred at the drift ratio of 9%, 8% and 7% for specimens SSK-P0, SSK-P1, SSK-P2, respectively, due to the fracture of the vertical reinforcing bars. The failure behavior of the specimens with RC shear keys was similar to that of the specimens with steel shear keys at and before drift ratio of 4%. After the drift ratio reached 5%, because the size of the RC shear key was larger than that of the steel shear key, it not only resisted shear forces, but also limit the rocking rotation of the segments. Thus, the RCSK specimens exhibited more hysteretic behavior and behaved more like the monolithic column specimen. Similar flexural-shear cracks that can be observed on the monolithic column specimen can also be observed on the west side and east side of the RCSK specimen. However, the extent of the diagonal crack for the RC shear key specimen was smaller.

All of these observed results given above confirm that the proposed precast segmental bridge columns integrated with the proposed semi-rigid connections can not only successfully emulate the stable ductility behavior of the conventional monolithic column constructed according to design code, it also suffered less damage and had a better ductility capacity.

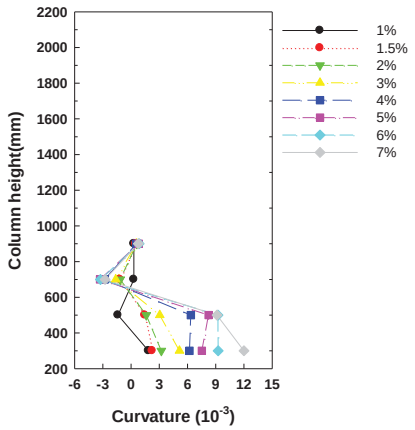
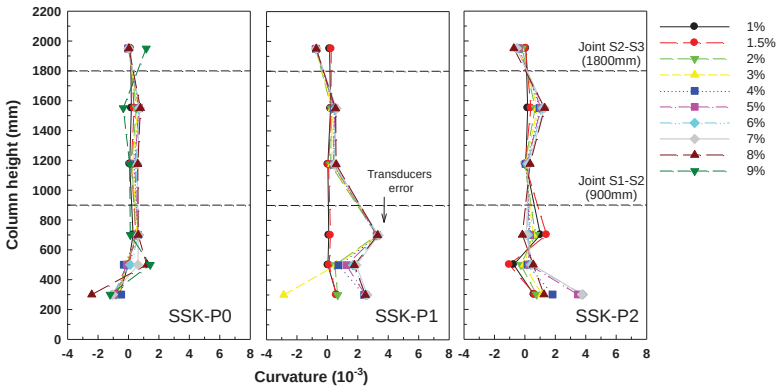
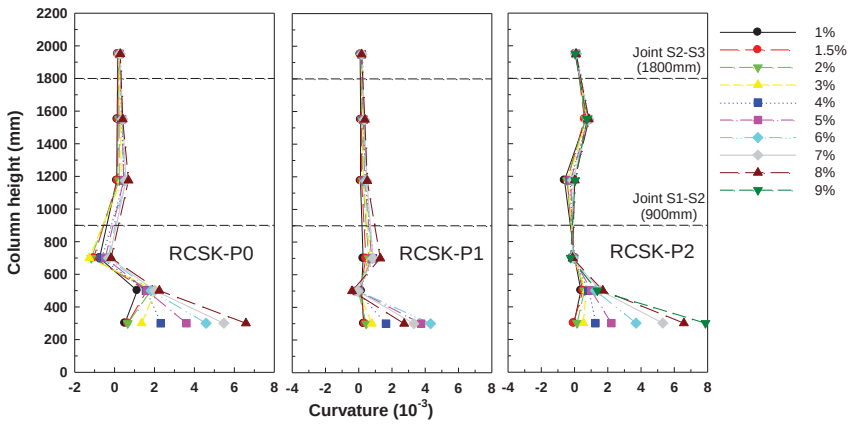


Fig. 8 Curvature distributions for specimen BM01 at different drift ratios (push direction, first cycle)



(a)



(b)

Fig. 9 Curvature distributions for specimens at different drift ratios (push direction, first cycle): (a) specimens SSK-P0,P1,P2 (b) specimens RCSK-P0,P1,P2

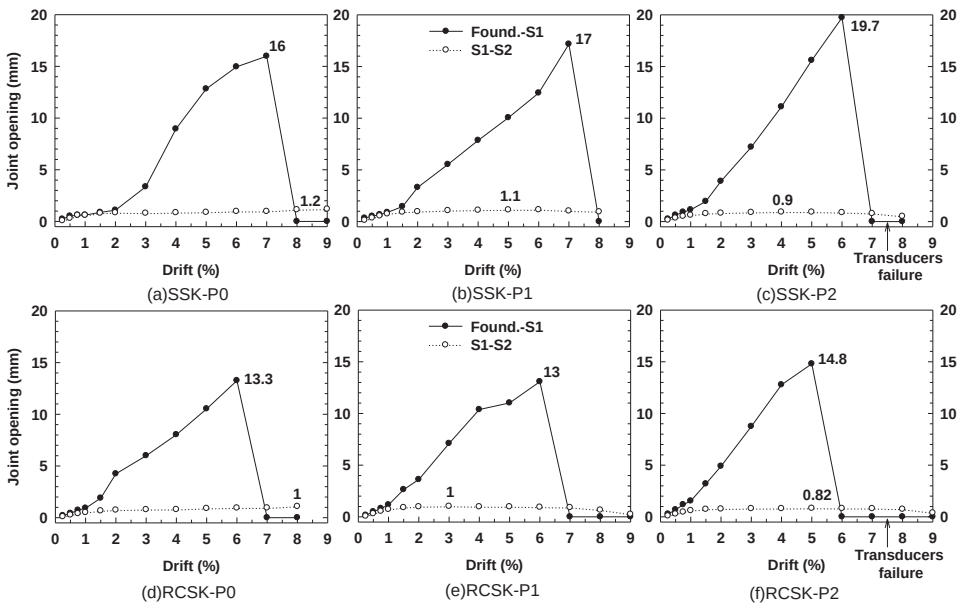


Fig. 10 joint opening

CONCLUSIONS

Six post-tensioned, precast segmental bridge columns with two different types of connection were tested under lateral cyclic loading to evaluate the seismic performance of the column details. The proposed connection between segments contains shear keys, unbounded prestressing tendons and bonded bar reinforcement that was spliced by bar couplers. The shear key can provide shear resistance between neighboring segments, therefore high prestress force in the tendon is not necessary. Only small amount of

prestress force is required for the unbounded prestressing tendons to provide re-centering force as the connection between the segments is open. In addition, the bonded bar reinforcement continuous between segments can provide the strength and energy dissipation capability. From the experimental results comparing with an equivalent monolithic cast-in-place column specimen and construction practices of the developed system, the stable ductility behavior that is similar to that of a conventional monolithic column and convenience of construction that is usually provided by precast segmental method were both confirmed.

ACKNOWLEDGMENT

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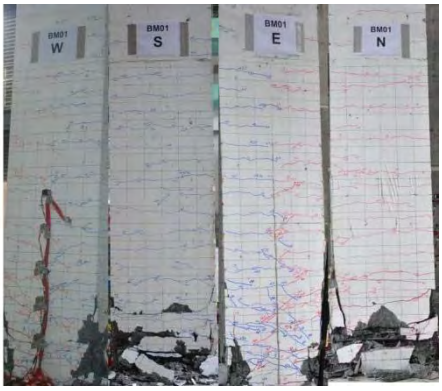
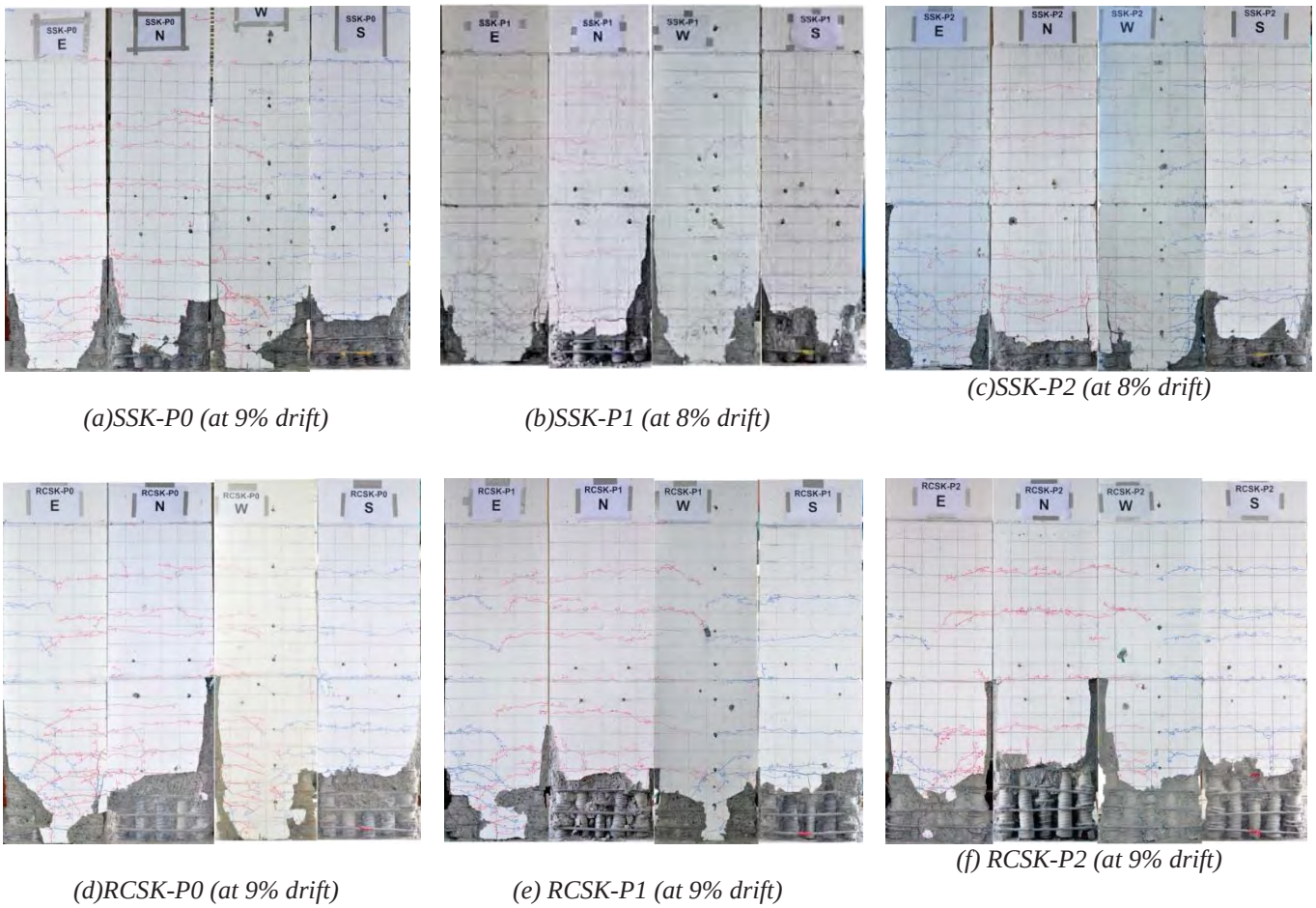


Fig. 11 Photographs of specimens BM01 after cyclic loading tests (at 7% drift)



(a)SSK-P0 (at 9% drift) *(b)SSK-P1 (at 8% drift)* *(c)SSK-P2 (at 8% drift)*

(d)RCSK-P0 (at 9% drift) *(e) RCSK-P1 (at 9% drift)* *(f) RCSK-P2 (at 9% drift)*

Fig. 12 Photographs of the precast segmental column specimens after cyclic loading tests

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PRECAST PIER CONSTRUCTION ON NORTHWEST CORRIDOR

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INTRODUCTION

The Northwest Corridor, the largest transportation project in Georgia's history, is a $599 million design-build project that includes 29.7 miles of reversible managed toll lanes and 39 bridges in metropolitan Atlanta. The project begins at the I-285 (Atlanta beltway)/I-75 interchange and parallels I-75 to the northwest and extends from the I-75 /I-575 Interchange and parallels I-575 to the northeast. The project scope of work includes design of roadway, bridges, utilities, drainage, signing, tolling facilities, retaining walls, sound barriers, intelligent transportation systems (ITS), lighting, environmental documentation and hydraulic in a densely populated urban environment. Existing I-75 Southbound currently accommodates 145,000 vehicles per day. As such, minimizing impacts to traffic during construction was a major consideration during design. Utilizing accelerated construction methods was one method used to help reduce impacts to traffic. Upon completion, the project will improve safety and mobility for the Atlanta Metropolitan Area.

This paper will highlight the innovative accelerated bridge construction (ABC) solution of piers on two project bridges.

BRIDGE 17

The first bridge is a 27-span I-75 toll bridge supported on hammerhead piers. The bridge centerline is located approximately parallel and centered between Hope Creek, and the existing SB I-75. The bridge substructure elements are to be constructed in the narrow wetland area between Hope Creek channel bank and the MSE wall that supports the existing I-75 Southbound roadway fill (See Figure 1). This area is environmentally sensitive with limited access. The MSE Wall height varies between 3.1 ft and 17.2 ft. The I-75 SB roadway is an extremely busy highway and consists of six (6) 12-foot travel lanes and an 8 ft outside shoulder.

The design/build team developed a special pier construction solution to minimize disturbance to the environmentally sensitive stream area, reduce impact to traffic, while improving construction schedule. A top/down construction method was utilized while restricting traffic only from the outside shoulder of I-75 southbound lane. The substructure is comprised of a pre-cast hammerhead pier caps supported on cast-in-place drilled shaft foundation. All construction activities will be performed behind I-75 southbound curb line. With selective crane positioning, the overreach distance from centerline column/caisson to the I-75 barrier curb and wall is limited to 7’-6”. A drilled shaft deep foundation solution was selected in-lieu-of a pile cap system to minimize excavation and disturbance to the wetlands while meeting the maximum overreach limit above. The weight of the pre-cast bent caps was carefully established based on allowable surcharge on the MSE wall and crane positioning. Given the overreach limit established, the precast cap weight during erection is limited to 150 kips. The pre-cast cap will be constructed in the field using the same bent cap forms used on other parts of the project. The cap/column connection was established using a chamfered rectangular blockout in the precast cap. Headed dowels confined by heavy transverse reinforcement will be used to reinforce this connection zone. A strut-and-tie model was developed to establish the distribution and extent of the column headed dowels into the cap. The pre-cast cap rebar was detailed to avoid conflict with the column headed dowels while the pre-cast cap is lowered into position. The column will be constructed to provide for a 6 in minimum to 12 in maximum bedding layer below the cap bottom. The bedding layer provides a relaxed construction tolerance for positioning the pre-cast cap. A temporary shoring will be erected on top of the column to support and provide geometry control to the pre-cast cap before casting the connection, (see Figure 1). The size and reinforcement detailing of the connection was designed such that standard strength concrete (f’c = 4500 psi) is utilized.

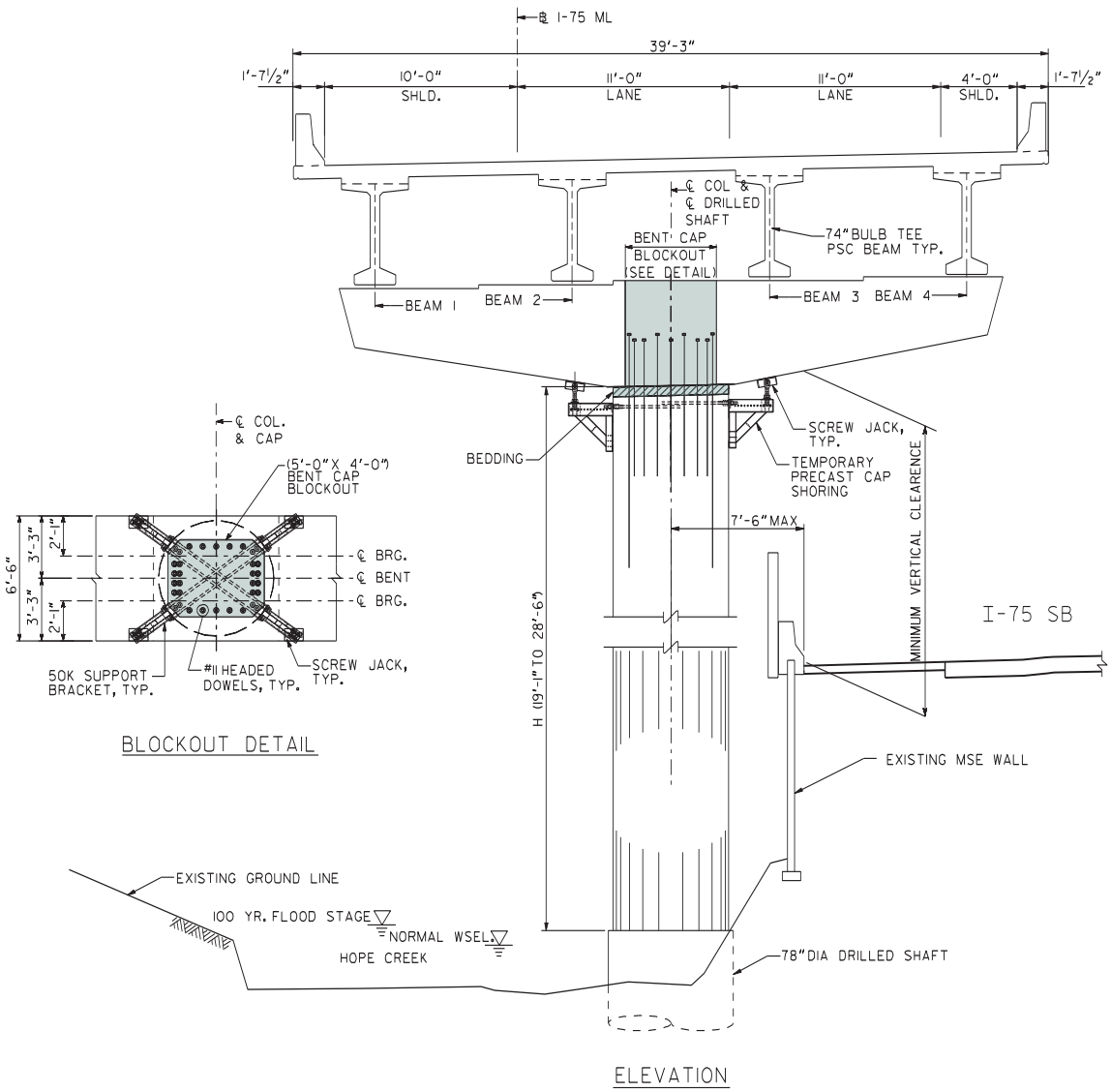

Figure 1. Bridge 17 Hammerhead Pier Precast Cap

During the cost estimate phase of this pursuit it became evident that access to this bridge site would be a major challenge. Several scenarios were proposed and priced of which two were deemed viable and compatible to maintenance of traffic (MOT) and environmental restrictions in the RFP. The two options that were considered was a conventional temporary work bridge (crane trestle) and ABC methods utilizing precast elements that would comply with the MOT restrictions. A subsurface investigation indicated the potential for difficult pile installation required for the trestle installation. This quickly made this option slow and cost prohibitive. Ultimately the scheme utilized drilled shafts and CIP round columns accessed from the shoulders and precast hammerhead caps erected during night time lane closures. The caps are to be cast on site and transported to the erection location via specialized trailers. Erection will be made utilizing a 300 metric ton mobile hydraulic crane placing the precast cap on temporary support

brackets. The same crane will also place the BT 74 prestressed concrete girders for the superstructure. The advantage to this type crane is the rapid setup/takedown required during lane closures and the ability to use varying counterweight packages and outrigger spreads to accommodate site and surcharge restrictions.

BRIDGE 11

The second bridge, a six-span I-75 mainline bridge over the existing I-75 southbound lanes will be constructed with very tight horizontal and vertical geometry controls, requiring construction of a straddle bent over three lanes of heavy traffic. In order to minimize traffic disruptions, a post-tensioned precast inverted tee pier cap and formwork will be erected over I-75 with limited, overnight lane closures. A second stage cast-in-place concrete cap will be constructed over traffic.

The precast section is 77’-0” long, with the core section 5’-0” wide weighing 125 tons. The precast bent cap will be cast on-site adjacent to the final bent location and post-tensioned on the ground. The precast cross section will have a “U” shape to allow seven ducts each with 19-0.6 inch strands stressed in a single stage post-tensioning operation. The “U” shape will also minimize the precast lifting weight and provides part of formwork for the cast-in-place concrete operation after the precast member is erected on the columns, (see Figure 2).

The bearing seats for the bridge longitudinal prestressed beams will be part of the precast concrete section, with 2’-2” wide ledges on each side of the precast section, (see Figure 2). The ledges reduced the effective prestress force in the beams, requiring considerable additional strands. To help stabilize the precast section after erection on the columns, the top of the 5’-0” column was increased to 8’-6” wide in the longitudinal direction. The precast cap width was similarly increased to 8’-6” in order to accommodate two bearings at each support.

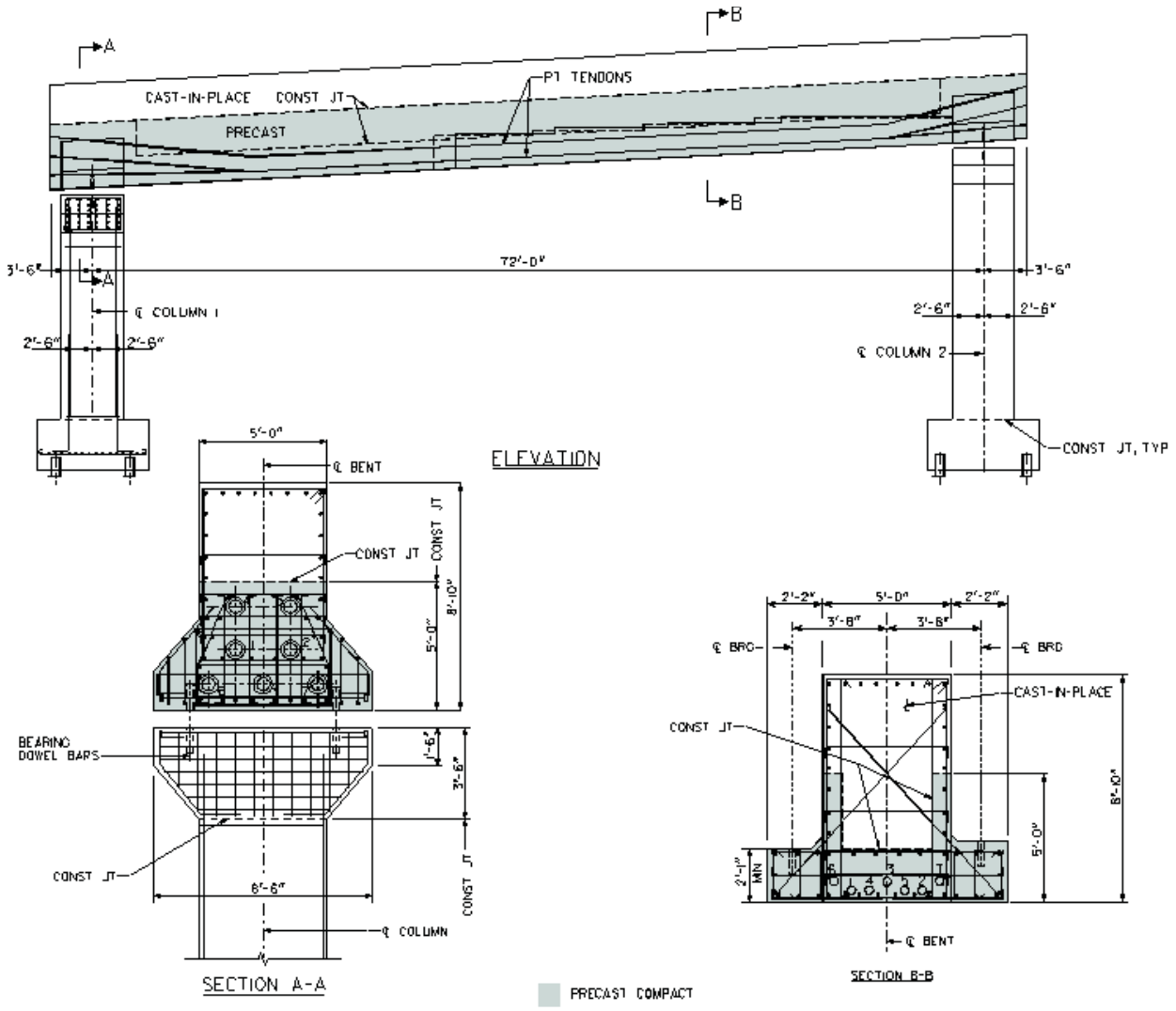

Figure 2. Bridge 11 Straddle Bent Precast Cap

G-6: RE-USE OF EXISTING SUBSTRUCTURES FOR ABC PROJECTS

BRIDGE SUPERSTRUCTURE-SUBSTRUCTURE INTEGRATED LOAD RATING

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ABSTRACT

Load rating of bridge superstructures is carried out regularly to determine their permissible load carrying capacity. In the current practice of load rating, the analytical mode of the bridges includes only the superstructure. In this research, an integrated approach to load rate bridges is proposed. In this approach, effect of settlement on the bridge superstructure is investigated to determine the maximum allowable settlement. Load rating criteria for the entire bridge structure are proposed in terms of maximum allowable settlement of bridge foundations.

SUBSTRUCTURE LOAD RATING APPROACH

Similar to the superstructure, substructure load rating can be done for Allowable Stress Rating (ASR), Load Factor Rating (LFR) and Load and Resistance Factor Rating (LRFR). The substructure load rating needs to be carried out at each applicable limit state and load effect for each of the three ratings.

As described above, by applying appropriate live load on bridge superstructure (e.g., HS-20 or HL-93 truck) and performing finite element nonlinear analysis, the pile head settlement is calculated for various levels of live loads. The results of this repetitive analysis are plotted as the transformed live load versus substructure settlement (displacement). Since the load bearing capacity of substructure depends on the mobilized settlement, the criterion of allowable substructure settlement becomes a key point. Limiting settlements for bridges depend on a number of factors including span lengths, geometry, material properties, etc., and can be calculated for a particular bridge based on the finite element analysis of a superstructure with live load and settlement. This limiting settlement, termed as “Maximum Tolerable Settlement” is discussed in the next section.

After defining the proper settlement criterion for the substructure, the corresponding transferred live load is extracted from load-settlement curves and the substructure rating factor therefore can be defined as follows,

$$RF_{sub} = \frac{C - DL}{LL_{tr}} \quad [1]$$

where C is bearing capacity of foundation, DL is the dead load and *LL* is transferred live load from superstructure to foundation. The rating tonnage is defined as rating factor multiply by gross weight of rating vehicle, i.e., $RT_{sub}=RFxW$.

Maximum Tolerable Settlement

The maximum tolerable settlement for a bridge may be defined as vertical settlement of the substructure under live load that will not cause stresses because of combined effects of live loads.and settlement, beyond allowable stresses in the superstructure. It is defined based on the live.load since the dead load generally causes uniform settlements. A number of research studies have been done on finding criteria for maximum tolerable settlements for substructures. Table 1 and 2 below summarize the existing literatures on the criteria for tolerable bridge settlement.

Table 2 Bridge tolerable differential settlement

Angular Distortion	Basis for recommendation	Reference
<0.004	Tolerable for continuous bridges	Moulton, et al. (1985) Duncan and Tan (1991)
<0.008	Tolerable for simple span bridges	Duncan and Tan (1991)

Table 1 Total bridge tolerable settlement

Maximum settlement	Basis for recommendation	Reference
2"	Not harmful	Bozozuk (1978)
2.5"	Ride quality	Walkinshaw (1978)
>2.5"	Structural distress	Walkinshaw (1978)
4"	Ride quality and structural distress	Grover (1978)
4"	Harmful but tolerable	Bozozuk (1978)
>4"	Usually intolerable	Whals (1990)

Strength Based Maximum Tolerable Settlement

For a simple span bridge, the tolerable settlement is serviceability limit state based rather than strength limit state. However, for continuous span bridges, the strength limit state becomes important. In reality, the maximum tolerable settlement for a bridge will be a complex function of superstructure geometry, material characteristics and span length and needs to be calculated for an individual bridge for a realistic assessment of substructure / superstructure load rating effects.

For a particular bridge, this can be calculated for each of the substructures through finite element analysis using superstructure strength information. In this approach, demands (over-moment and over-shear) due to different values of settlement for Strength load combination 1 (SLC 1) using linear elastic FE analyses are calculated. It should be noted that the maximum moment due to settlement always acts at a pier location. The moment and shear envelope with zero settlement for Strength I load combination is also calculated to account for live load effects. Then, total demands (because of settlements and live loads) are compared to resistance (strength) to calculate the settlement limit through interpolation. Figure 1 shows the flow chart for the calculation of maximum tolerable settlement for substructures.

Superstructure-Substructure integrated load rating

From load carrying point of view, the superstructure and substructure elements must be compatible. The loads placed on the bridge superstructure should not exceed the load-carrying capacity of the substructure. The load rating of bridge superstructure may therefore depend on substructure load rating for some cases. In general there are two types of factors which require substructure load rating: any change in the transferred load from superstructure to substructure and whatever reduces load bearing capacity of foundation. Table3 below represent the possible when superstructure only or integrated superstructure-substructure load rating may be applicable.

It should be noted that the cases presented in Table 3 are based on qualitative investigation and need to be investigated further.

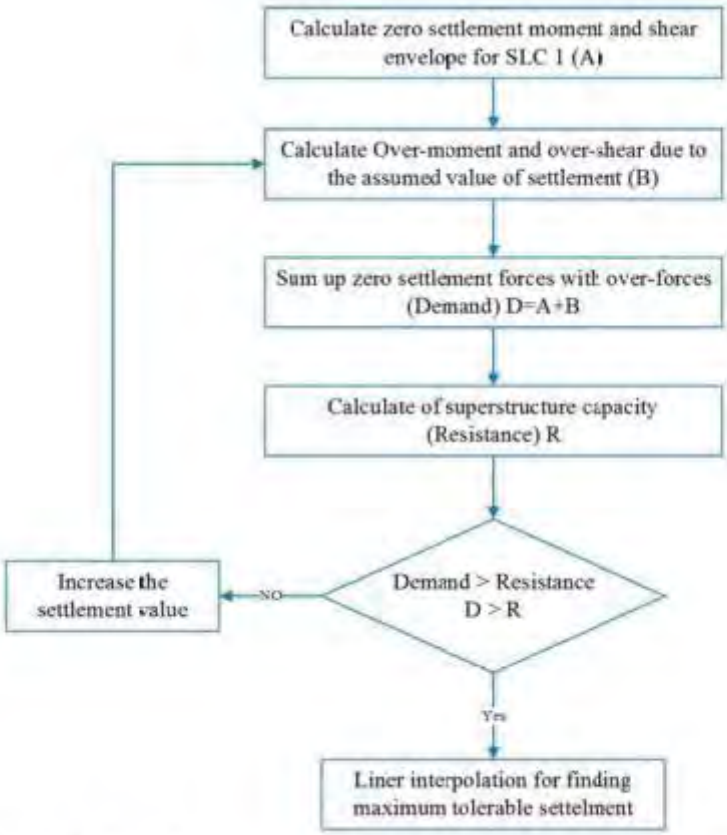

Figure 1 Strength-based maximum tolerable settlement calculation flowchart

Table 3 Superstructure load rating versus integrated superstructure-substructure load rating

Case	Superstructure Load Rating	Integrated Load Rating
Dead Load Changes		
Re-decking	✓	
Deck widening		✓
Girder addition / replacement		✓
Accidental superstructure damages	✓	
Minor Rehabilitation (seismic, corrosion and repair)	✓	
Fracture Critical Bridges		✓
Live Load Changes		
Traffic pattern change		✓
Live load specification changes		
High loads		✓
Scour Critical Bridges		✓
Substructure Post Hazard Affected		✓
Substructure Retaliation/Retrofitting		✓
Change in analysis methods	×	×
Foundation Reuse		✓

Figure 2 shows the flowchart for the integrated superstructure – substructure load rating. The superstructure load rating with vertically constrained supports is the first step of this procedure. This is followed by the calculation of maximum tolerable settlement for the superstructure. If the bridge isn't scour critical, then the settlement of the substructure under the load transferred from the superstructure under the rating load (e.g., HS-20) is compared with the maximum tolerable settlement. If the settlement of the substructure is less than the maximum tolerable settlement, then the load rating of the bridge is equal to the superstructure load rating. Otherwise, bridge load rating will be smaller than the superstructure load rating and will be dominated by the substructure condition. This lower rating will be calculated from the load-settlement curve for the substructure corresponding to the maximum tolerable settlement for the superstructure.

For bridges prone to scour related settlement, settlement of substructures caused by combined effects of dead load and scour need to be calculated. If this settlement is less than the maximum tolerable settlement of the superstructure, then the calculation above for the substructure load scour countermeasures to limit settlement and then perform the substructure load rating calculations or the bridge needs to be replaced.

Overall, the integrated superstructure-substructure load rating approach in the Flowchart in Figure 2 presents an approach that can be used to define load capacity of the bridge by including any kind of foundation effects. This load rating may not be necessary for all bridges. Rather, it may be applicable to

some bridges where foundation load capacity dominates. This aspect is being investigated through case study on select number of bridges.

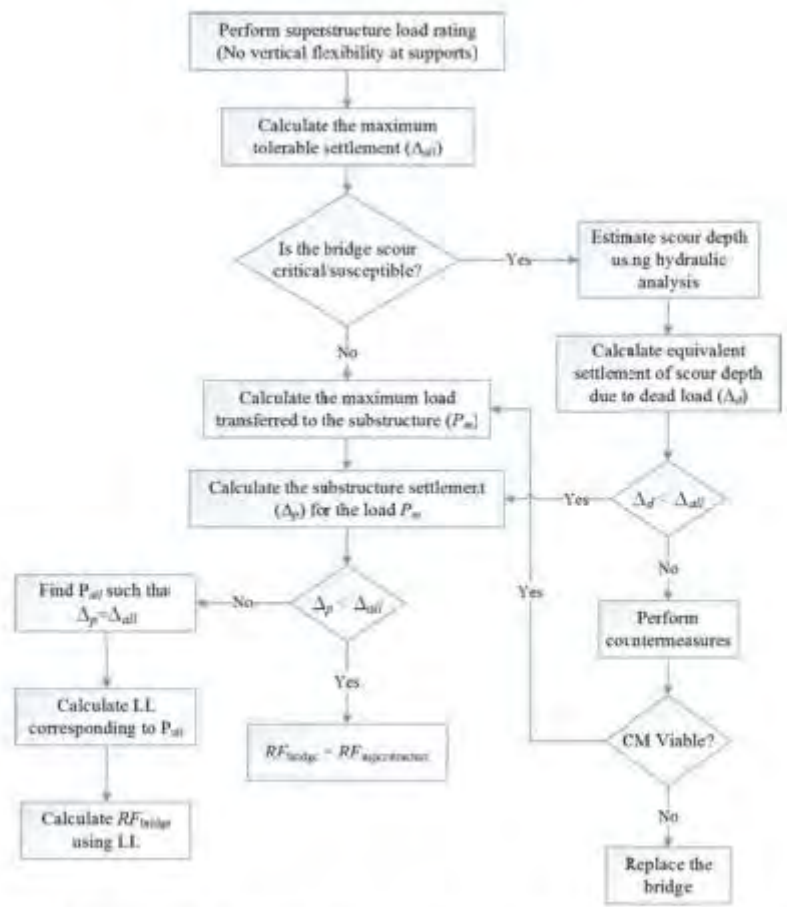

Figure 2 Integrated superstructure-substructure load rating flowchart

Case Studies

Several example bridges have been considered to investigate the integrated superstructure- substructure load rating of bridges using finite element approach. Applications of the proposed approach to calculate load rating of bridge foundations will be presented during the conference.

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FOUNDATION CHARACTERIZATION AND REUSE IN ABC PROJECTS

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BACKGROUND

The reuse of bridge foundations in ABC projects is gaining popularity in order to save time and money. However, there are many challenges for the reuse of existing foundations. One of the major challenges is to characterize the foundations of existing bridges for durability, integrity and load capacity. This information is critical in bridge owner's decision making process for determining whether they can rely on the existing foundations to continue to carry increasingly heavier loads; and/or to withstand future geotechnical and hydraulic hazards.

As part of the Federal Highway Administration (FHWA) foundation characterization research program (FCP), the Willow Valley Bridge was selected as a field site for the assessment of existing and development of new technologies for foundation characterization and reuse. The bridge is located 40 miles (64 kilometers) southeast of Flagstaff, AZ, on I–17 along the low-volume Lake Mary Road. The proposed rehabilitation plan proposed by FHWA's Central Federal Lands Highway Division is to replace and widen the existing bridge superstructure.

As shown in figure 1, the existing three-span Willow Valley Bridge is 104-foot (31.7 m) long and 34-foot (10.4 m) wide. The bridge was constructed in two phases: the original construction of the southern portion in 1934 and a widened portion in the northern portion in 1968. The bridge widening added three more girder lines. Mass-gravity substructure supports the pin-and-hanger steel girders and concrete deck superstructure. Each stone masonry wall foundation is founded on rock.

To implement the reuse of foundation concept and assure a safe and stable structural design, the existing foundations was thoroughly evaluated. Extensive data were taken for this study; a small sampling is presented herein to highlight the capability of technologies.

Figure 1. Several views of the Willow Valley Bridge.

WIRELINE LOGGING FROM COREHOLE

For the purpose of this study, two 3-in (7.6 cm) diameter core holes were drilled at each foundation wall and were advanced from the bridge deck through the mass gravity columns into the bedrock. Wireline logging technology was used to assess mapping the crack, present Shear/Young's modulus as a function of depth, and address durability issues of the wall.

Figure 2 indicates optical televiewer plot. The tool generates a continuous oriented 360° image of the corehole wall using an optical imaging system. The tool has a built in 3-axis magnetometer and 3-axis accelerometer that correct for its orientation and create a "virtual core" image of the core hole wall.

Figure 2 shows both the full 3D corehole image on the left and unwrapped close-up image near the bedrock on the right. The unwrapped close-up image on the right indicates voided areas with very low core recovery rate of 50%. Additionally, crack planes and dip directions can be picked.

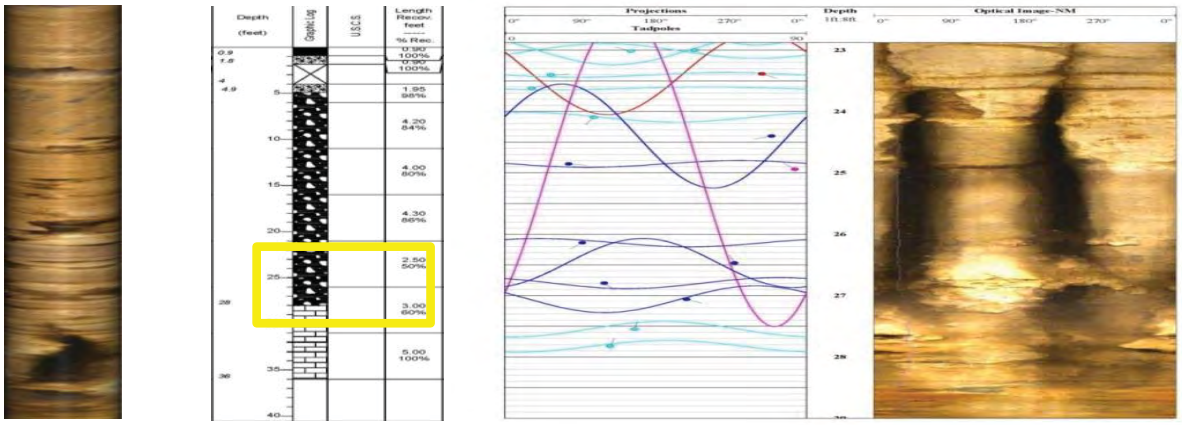

Figure 2 – Optical televiewer image for Abutment 1 from Corehole B1 indicating the full 3D oriented image (left); field corehole log data (center); and unwrapped televiewer close-up image between 23-28 ft (7-8.5 m) indicating open voids and dip direction and angles (right two images.)

GEOPHYSICAL VOLUMETRIC IMAGING

Wireline logging can provide detailed information on deteriotation of the foundation material around a corehole. Other geophysical imaging technologies can produce three dimensional volumetric images for assessing structural integrity. Figure 3 shows a velocity "tomogram" that indicates the distribution of acoustic velocity in the wall with low velocities indicating possible anomalies The velocity tomogram was then used to obtain "reflectogram" image of the echoed wave arrivals. The imaged waveforms correlated well, not only with the known abutment wall boundaries, but also with the interpreted location of voids.

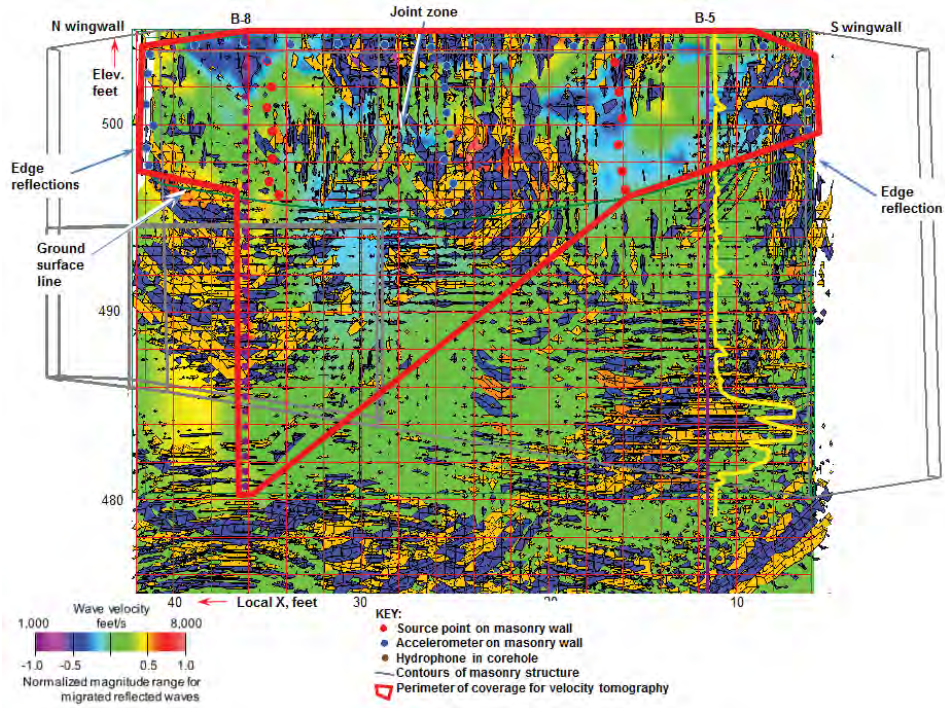

Figure 3 –Color Tomogram of Abutment 1 indicating velocity distribution with reflection echoes superimposed. Red line marks perimeter of seismic survey coverage.

The results obtained by this study assisted the FHWA Central Federal Land office in developing a plan to grout the voids in the foundation walls in assessing the structural integrity and capacity of the foundation wall. Although the degree and complexity of the surveys obtained by this study is not needed for every foundation reuse project, one goal of the FHWA's foundation characterization research program (FCP) is to deploy technology at a reasonable cost and the development of a guidance and best practice document for foundation reuse.

ASSESSEMENT, RISK MANAGEMENT AND REHABILITATION OF EXISTING STRUCTURAL FOUNDATIONS

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ABSTRACT

This presentation will address the program, process and project issues associated with the assessment, risk management and rehabilitation (reuse and enhancement) of existing structural foundations (both shallow and deep). The reuse of existing structural foundations, both shallow and deep, is an emerging technical subject which provides significant opportunities (benefits of cost and schedule) as well as risks (compatibility, long term reliability and costs) for the highway engineering and construction community. The materials and techniques which may be applied to underpin and enhance the structural and geotechnical resistance of an existing foundation include micropiles, augercast piles, drilled shafts and several ground improvement methods including grouting, and controlled modulus columns. These technologies are at various levels of maturity; many have well developed design, construction and monitoring protocols yet some are new, some are old or not used routinely at a local or regional levels.

The application of these techniques however for the reuse and enhancement of existing structural foundations present unique design and construction challenges including deformation limit requirements, compatibility with the existing foundations, highly restrictive noise and vibration criteria, and difficult and limited access during construction. Finally, some technologies have only limited qualitative methods of design verification and other technologies have verification methods which are costly and time consuming.

Use of existing foundations (with or without enhancements) is not applicable for all projects because of geometric constraints or current design and performance requirements. However, when applicable the time and cost benefits may be significant and can often can be accomplished with confidence.

Each technology will be briefly “introduced” for reference and then the following technical topics will addressed with a focus on existing foundation enhancement:

- Definition, material components basic construction process
- Level of technology maturity
- Advantages and limitations
- Recent advancements
- Applicable soil and rock materials
- Resistance capability to compression, uplift, lateral loading
- Vertical and lateral deformation response
- Available test methods for quality control during construction and integrity and geotechnical quality assurance tests following installation

The state of the art and state of practice for structural foundations and ground improvement technologies have significantly advanced during the past 40 years. Modern deterministic or reliability based design methods and foundation software tools can produce accurate predictions of foundation performance and vertical and lateral deformations as new loads and moments are applied across the composite (existing and enhanced) foundations. More accurate and precise load-deformation prediction models for geomaterials extend beyond the application of foundation reuse and may be applied on new facilities. Traditionally information on foundation performance was not well communicated between the geotechnical and structural disciplines. Today, improved communication between technical disciplines correctly focuses the many variables which affect foundation performance and costs. For example, post construction vertical and lateral deformations are more closely examined in terms of the time and incremental application of both the transient and permanent loads. In addition attention is correctly drawn to the location of deformation interest (ex. top of the pier or beam seat of an abutment). Although isolated examples of model best practice exist unfortunately the mainstream application use of these approaches are yet far from being mainstream.

The foundations of a significant number of existing structures were over designed and constructed by today's standards. As a result these foundation systems were often installed with considerable (yet unaccounted for) reserve geotechnical and structural resistance. The questions for today's designer who wishes to consider reuse of the existing foundations are:

- What is the material condition of the existing foundation elements (environmental deterioration)?
- Were the originally designed foundation elements damaged during installation (installation defects and overdriving)?
- What is current the nominal structural and geotechnical resistances of the individual elements?

- Finally if the existing foundation is inadequate to provide the required structural and geotechnical resistance, then what technologies might be technically feasible and cost effective to enhance its performance?

These rather straightforward questions are often difficult to answer at a programmatic level and sometimes on individual projects without considerable study and investigation. Other non-technical issues must also be considered:

- What risks are the owner and the designer willing to accept and share regarding the technology being used and the end product which might only be able to be verified on a qualitative basis
- What is a reasonable standard of care for such an evaluation and what liability is the new structure designer assuming by assigning a defined service life to a “new” structure?

This presentation will also address considerations for the selection and application of technically appropriate and cost effective “underpinning and enhancements” of existing foundations, from both a load and resistance design perspective. The presenter will address the application of existing AASHTO Bridge Design and Construction Specifications, as well as Federal Highway Administration and national industry recommended guidance. Selection of the appropriate solution is similar to a new project but also includes unique considerations including design/ service life of the “enhanced” foundation.

Each project has unique and specific requirements and constraints and application of these materials and techniques may not always be appropriate. When appropriate, however, they collectively provide significant and measurable benefits in terms of improved performance and savings of time and cost.

G-7: ADVANCING SEISMIC RESEARCH FOR ABC

ACHIEVING CALTRANS' STRATEGIC GOALS THROUGH ACCELERATED BRIDGE CONSTRUCTION

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INTRODUCTION

In an effort to address a mounting number of transportation challenges that impact California's bridges and structures, Caltrans developed the California Bridges and Structures Strategic Direction. The

Strategic Direction is an integrated management plan to insure Californians have safe, sustainable, cost effective transportation structures that are well-built and maintained in compliance with all applicable regulations.

The Strategic Direction is a collaborative effort to manage these assets independent of ownership or funding sources. The Strategic Direction identifies 12 objectives that maximize innovation, sustainability, integrated planning, design, construction, and maintenance of bridges and structures. It is becoming increasingly more evident that Accelerated Bridge Construction (ABC) is an essential tool for California to address its transportation challenges and achieve its strategic objectives.

SEISMIC SAFETY WITH ABC

Safety is the number one strategic goal at Caltrans. Safety encompasses not only motorists and workers during normal highway operation; it also includes the safety of the public during and after an extreme event.

In addition to meeting the minimum Caltrans 'no collapse' seismic performance requirement, bridges built with ABC techniques also must adhere to the basic principles of Caltrans' displacement-based seismic design philosophy; ductility, energy dissipation, capacity protected design, redundancy, and pre-determined locations of damage. Initially these principles posed challenges in how we connect pre-fabricated structural components. To avoid these challenges, Caltrans' initial ABC projects focused on precast abutments and simple span slide-in construction. In 2008 Caltrans launched a multi-phased research program to develop seismically resilient connection details for pre-fabricated elements. This research program has recently been completed and a suite of component connection details will be deployed in an upcoming pilot program.

In conjunction with the multi-span pilot projects, Caltrans is initiating a system test of a scaled standard bridge using ABC methods. The test specimens will utilize previously vetted precast components in order to assess constructability, service performance, and expected seismic performance. The research includes investigating full-depth replaceable precast modular deck systems that may reduce the life cycle cost of future bridges.

Performance Based Earthquake Engineering

In addition to safety, Caltrans has identified system performance as one of its top five strategic goals.

With the completion on of the legislated bridge seismic retrofit programs, our attention is focusing on strategically investing resources and capital to maximize seismic risk reduction on a system wide basis.

Caltrans has been researching and developing Performance Based Earthquake Engineering (PBEE)tools and methods to accomplish this goal. Currently, Caltrans uses a single life safety or no collapse performance criteria for ordinary bridges.

Caltrans is exploring PBEE as a means to initiate multi-level performance criteria. Increasing the resiliency of the bridge inventory will reduce the geographical area of earthquake damage thereby reducing societal losses caused by a seismically impaired transportation system. Bridge resiliency has a direct correlation to life safety and accelerated economic recovery after an extreme event. Caltrans is developing ABC techniques coupled with seismic response modification devices to efficiently improve system performance after earthquakes. Caltrans has also renewed its interest in developing other resilient technologies well suited to ABC construction such as self-centering bridge systems and replaceable seismic fuses that can be replaced rapidly after an earthquake that will hasten the restoration of the highway system.

Caltrans has recently completed a seismic screening of the state bridge inventory that incorporates a variety of seismic hazards, most of which were not considered in previous retrofit screenings. The results of the screening is a risk based seismic vulnerability prioritization of the bridge inventory that can be combined with other bridge condition and life cycle analysis to prioritize overall system risk. Many of the bridges likely to be targeted as high risk are older bridges on heavily traveled freeways in urban areas.

These bridges typically have been widened multiple times with various structural systems and often carry more than ten lanes of traffic. Historically Caltrans has replaced these type of bridges in multiple stages that can take years to complete. Seismically robust multi-span ABC methods will enable Caltrans to replace these bridges more efficiently while lessening the impact on the traveling public.

Rapid Renewal

More than 1600 state bridges experienced threatening levels of horizontal ground motion during the 1994

M 6.7 Northridge Earthquake. Of the 2523 state bridges located in Los Angeles County at the time of the earthquake, seven bridges collapsed, 39 had major damage, and 194 had moderate or minor damage. Considering that Los Angeles County now contains over 5000 state and local agency bridges, the L.A. basin may have a much more significant and broader damage scenario after a 975-year design level earthquake. If such a scenario occurs, the traditional methods of construction coupled with accelerated contracting methods such as incentive/disincentive and cost plus time bidding will not be sufficient to return the highway network to a reasonable level of serviceability in an acceptable timeframe. Pre-engineered,pre-fabricated component based designs can provide a means to rapidly manufacture bridge components in parallel with bridge demolition and site preparation work to restore system performance and stimulate economic recovery.

ENVIRONMENTAL RISK MANAGEMENT WITH ABC

Sustainability and environmental stewardship are not only strategic objectives for Caltrans, they are executive orders by California's Governor. To date, the majority of Caltrans bridge projects utilizing ABC have been in response to environmental mitigation requirements. This trend will continue as regulatory agencies realize the environmental benefit of smaller construction foot prints and shorter duration on site for bridges built with ABC techniques. Currently Caltrans is utilizing lateral slide technology to reduce the environmental impact of a major bridge replacement over a federally protect wild and scenic river.

Similarly, a modular galvanized steel tied arch is under design to span over a protected beaver pond on Native American tribal land. ABC techniques are so effective in mitigating environmental impacts they are driving type selection decisions that often lead to non-traditional bridge types for California.

Recent California legislation prohibits extending the life of any culvert that is identified as a barrier to fish passage or constructing projects that are barriers to fish passage. Caltrans has identified 563 locations where fish passage needs to be restored. Caltrans is working with the regulatory agencies on developing pre-engineered ABC details that can be approved programmatically, thereby streamlining the environmentally process as well as minimizing the impacts to the streambeds during construction.

ADDRESSING BRIDGE MAINTENANCE WITH ABC

Stewardship extends to asset management. Although Caltrans has a “fix-it-first” mentality, recent projections show Caltrans will need approximately $8 billion annually to fund necessary improvements and preventative maintenance over the next ten years. Projections show available revenue is $3.3 billion annually. Efforts are underway to close the $5.7 billion shortfall so we can reverse the trend of deterioration that leads to more expensive remedies in the future. Caltrans is utilizing ABC methods “to get in, get out, and stay out”. Deploying ABC methods on bridge maintenance projects can maximize the effectiveness of our rehabilitation funding as well as reduce worker and motorist exposure to injuries and fatalities. Caltrans recently completed a multiple hinge replacement project utilizing rapid strength concrete. Currently Caltrans is studying rapid strength concrete and full depth concrete precast deck panels to replace a 65 year old deck on one of the busiest interchanges in the San Francisco Bay Area.

A significant percentage of state owned bridges in California are post tensioned box girders. Cast-in-place post tensioned (CIP/PT) box girder construction expanded rapidly in the mid to late 70's and continues. As the post tensioned box girder bridges age, the bridge decks need ongoing preservation and in some cases replacement. Currently there is no way to replace the deck of a post tensioned box girder bridge in service. Design and/or construction methodologies are needed for the replacement of deteriorated post tensioned box girder decks without the reliance on falsework while maintaining reduced traffic service on the bridge. A research project has recently been started to develop methods to replace CIP/PT bridge decks. While the outcome of the research is unknown, it is anticipated that ABC methods will be necessary to reduce the cumulative time required to replace entire decks on box girder bridges.

CONCLUSION

Caltrans has steadily increased its deployment of ABC. While motorist and worker safety remain our number one objective, ABC has and will continue to contribute to Caltrans’ efforts in protecting the environment, improving the seismic safety of our transportation network, and promoting cost effective bridge maintenance solutions. These goals are the essence of Caltrans mission to provide a safe, sustainable, integrated and efficient transportation system to enhance California’s economy and livability.

SEISMIC EVALUATION OF A PRECAST PT/UHPC BRIDGE COLUMN WITH POCKET CONNECTION AND PRECAST FOOTING

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ABSTRACT

Seismic response of a posttensioned precast bridge column connected to a precast footing using pocket connection was investigated in this study. The objective was to develop a damage free precast bridge column with reliable and resilient connection. The precast column was post tensioned with unbonded carbon fiber reinforced polymer (CFRP) tendons to eliminate permanent drift. Furthermore, ultra-high performance concrete (UHPC) was used in the plastic hinge of the bridge column to reduce damage under the design earthquake and as grout in the pocket connection to improve bond between the prefabricated column and the footing. Shake table test was conducted to failure. Results showed that the pocket connection performed well without any damage in the pocket area and footing and negligible permanent drift. The embedment length of the column was adequate to provide full fixity at the base. In addition, CFRP tendons effectively eliminated the residual displacement at different levels of earthquake.

INTRODUCTION

Accelerated bridge construction (ABC) has recently become popular due to its numerous advantages such as minimizing traffic delays and road closures, as well as reducing the construction time and efforts.

Prefabrication of bridge components are essential for ABC to succeed. Connections of the prefabricate members are particularly critical in moderate and high seismic zones. Structural integrity of the bridge has to be maintained by capacity protected connections that experience no or little damage under service loads and extreme events. One of the methods to connect prefabricated bridge columns to footings is pocket connections. This study focused on these types of connections as they have shown promising results while not violating the current AASHTO and Caltrans seismic codes. The main objectives of this study were: (1) to evaluate the seismic performance of pocket connections for square columns, (2) determine the effectiveness of ultra-high performance concrete (UHPC) in the plastic hinge zone in

minimizing column damage, and (3) to investigate effectiveness of unbonded carbon fiber reinforced polymer (CFRP) tendons in recentering columns under strong earthquakes.

INNOVATION CONCEPTS

Pocket Connection

Seismic response of pocket connections has been studied by several researchers to determine the required embedment length of the columns in the footings. Kavianipour et al. (1) and Motaref et al. (2) tested bridge piers having pocket connections at footing under seismic loading. The embedment length of the columns at the pocket connections was 1.5 times the diameter of the columns in both studies.

Authors reported that the embedment length was sufficient to develop full fixity at the base of the columns. Haraldsson et al (3) tested bridge columns having pocket connection at footing with the embedment length of 1.1 times the columns diameter under cyclic loading. Authors reported that the embedment length was sufficient to protect the footing by promoting failure in the column, as is required in bridge design. The cross section of the bridge columns tested above was circular. Since there was a lack of information on pocket connections with square columns, the cross section of the novel column was square in this study. The embedment length was 1.0 times the column dimension. Figures 1.a-b shows the precast column, footing, and the pocket connection.

Ultra High Performance Concrete (UHPC)

UHPC exhibits mechanical and durability properties that make it an ideal candidate for use in developing new solutions to pressing concerns about highway infrastructure deterioration, repair, and replacement

(4). UHPC is a cementitious material with water-to-cementitious material ratio of less than 25% while having a high percentage of steel fiber reinforcements. The 28-day compressive strength of UHPC is greater than 20 ksi and the tensile strength is greater than 0.7 ksi. The mechanical properties of UHPC make it attractive for use in the plastic hinge zone of the bridge columns to reduce seismic damage.

Therefore, UHPC was used in the plastic hinge of the column from footing interface to 2.0 times the column dimension above the footing.

CFRP Tendons

CFRP tendons have been used in lieu of steel tendons in concrete structures (5). The promise of CFRP tendons relies in their noncorrosive, nonmagnetic, high strength, and light weight properties. Field applications of unbonded posttensioned steel tendons have not been widespread in bridge columns because of concerns about corrosion of steel tendons. To eliminate concerns about corrosion, CFRP

tendons were utilized instead of conventional steel tendon in this study. Figure 1.c shows CFRP tendons used in this research project. Effective cross sectional area of CFRP tendons was 0.88 in and the guaranteed tensile strength of each tendon was 270 kips with the elastic modulus of 21,030 ksi.

Figure 1. Pocket connection at footing (a & b), CFRP tendons (c)

EXPERIMENTAL STUDY

To accomplish the objectives of the study, a 1/3 scale of a precast novel column model was constructed and posttensioned using CFRP tendons. The plastic hinge zone was made with UHPC. The column was inserted in the precast footing, which had a pocket area in the middle for the column. Afterwards, UHPC was used to fill the gap between the column and the footing. Column dimension was 20”x20” with an aspect ratio of 4. The longitudinal steel ratio was 1.2% and transverse steel ratio was 1.6%. Spreader beam was used to apply 100 kips of axial load and the initial posttensioning force was 128 kips. Total axial load index considering dead load and the initial posttensioning force was 14.25%. Figure 2 shows the test set up. This column was designed according to AASHTO guide spec (6) assuming the bridge was located in the Los Angeles area at Lake Wood, with the latitude and longitude of 3.84926 N, and

118.09252 W, respectively. Seismic properties of this location were as follows: As=0.473g, SDS=1.155g,

SD1=0.637g, To=0.11 sec, Ts=0.552 sec, Site class: D. The 1994 Northridge earthquake acceleration history recorded at the Rinaldi station was simulated in the shake table test because of its tendency to cause large permanent displacements in conventional reinforced concrete columns. A total of six runs,

25%, 50%, 100%, 133%, 167%, and 200% design level were applied in the shake table test to capture the seismic response of the novel column under different levels of earthquake. Figure 3 shows the hysteresis loops of the column during the six runs. According to the experimental results, the novel column reached approximately 7% drift and the residual displacement was nearly zero at the end of each run. Figure 4 shows the damage state in the loading direction, north-south, of the column at the plastic hinge and the

pocket connection after the 200% design level earthquake was applied. According to the test observations, the pocket connection performed well without any considerable damage at the pocket area and the footing.

CONCLUSIONS

According to the shake table test of the novel column, the pocket connection performed well and the embedment length of 1.0 times the column dimension was adequate to provide full fixity at the base.

UHPC in the plastic hinge eliminated the seismic damage and concrete spalling. Due to the high compressive strength of UHPC, the column failure mode was rebar rupture rather than core concrete failure, meaning that the column reached the maximum flexural capacity. In addition, CFRP tendons effectively eliminated the residual drift during different levels of earthquakes and can be used as a replacement for the steel tendons in bridge columns.

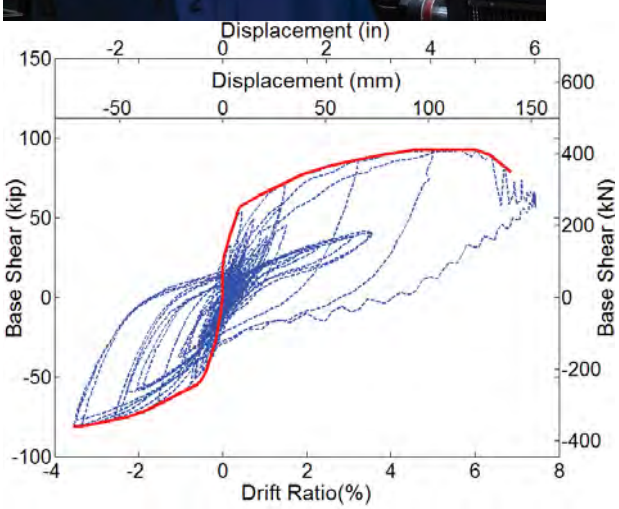

Figure 2. Test setup of the novel column

Figure 3. Hysteresis loops of the novel column

Figure 4. Damage state of the novel column after 200% design earthquake

ACKNOWLEDGEMENT

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SEISMIC REPAIR OF DAMAGED PRECAST RC BRIDGE COLUMNS CONNECTED WITH GROUTED SPLICE SLEEVES

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ABSTRACT

A repair technique for damaged precast reinforced concrete (RC) bridge columns connected with grouted splice sleeves (GSS) has been developed that utilizes a carbon fiber-reinforced polymer (CFRP) shell and epoxy anchored headed mild steel bars to relocate the column plastic hinge. Four original specimens were built using an

Accelerated Bridge Construction (ABC) technique with two different GSS systems and tested to failure using cyclic quasi-static loads. One GSS system was used to connect an RC bridge pier cap to a column and the second GSS system was used to connect an RC footing to a column, simulating an ABC bridge bent. Failure of the four original specimens occurred at drift ratios between 6.0% and 8.0% with longitudinal bar fracture or pullout from the GSS connections. Both normal strength nonshrink concrete and expansive concrete were used for repairing the pier capto column and footing-to-column connections. The use of expansive in place of nonshrink concrete caused sufficient dilation to produce an active confinement system. Due to the active confinement, the CFRP shell was engaged from the start of the experiment and did not require initial cracking of the repair concrete to engage. The additional confining pressure gained with active confinement increased the concrete tensile capacity which helped negate transverse CFRP shell cracking and demand on the headed bars. The repair method successfully relocated the plastic hinge to the original column section adjacent to the repair and was capable of restoring the load and displacement capacity. The method is a viable and cost-effective technique for rapid seismic repair of precast bridge assemblies.

INTRODUCTION

Repair of severely damaged bridge elements following an earthquake is an advantageous alternative to replacement. The benefits include cost savings, reduction in construction time and decreased interruption of emergency services. The objective of bridge repair is to rehabilitate the damaged bridge elements to a

performance level similar to the original performance by restoring the load and displacement capacity of the system. Repair techniques for damaged bridge columns include the use of externally bonded Carbon Fiber-Reinforced Polymer (CFRP) jackets (1-6), steel jackets (7-9) and concrete jackets (10, 11). However, until recently it has been assumed that when longitudinal bars within the column buckle or fracture the column should be replaced (12).

Accelerated Bridge Construction (ABC) is gaining acceptance because of reduced construction time and minimal traffic interruption. Grouted Splice Sleeves (GSS) have been gaining attention as a possible precast concrete connection method for ABC in seismic regions. Current research is focused on the performance of GSS connections for bridges built in seismic regions (13-16). The use of GSS connections in moderate to high seismic regions is imminent and a practical post-earthquake repair method is needed to accompany this new technology. Findings from current ABC research indicate that columns connected using GSS concentrate column damage and decrease the plastic hinge length compared to traditional monolithic construction (17). These damage characteristics are advantageous for repair purposes, leaving a column section with minor damage for plastic hinge relocation.

The repair method developed has been designed and implemented on four severely damaged precast specimens connected using GSS. The original specimens had undergone quasi-static cyclic testing and had reached a severe damage state, before being repaired. The repair uses materials that are available and easy to install including epoxy anchored headed bars, CFRP sheets and nonshrink or expansive concrete (18). The result is a cost effective, corrosion resistant, rapid repair procedure which could be installed in a few days. Due to the robust nature of the repair it is a suitable option for columns with varying damage states, including columns with buckled or fractured longitudinal bars, presented in this paper.

EXPERIMENTAL INVESTIGATION OF ORIGINAL SPECIMENS

Original Test Specimens

Four precast RC specimens representing half-scale bridge elements, conforming to current seismic bridge design standards, were constructed utilizing two different GSS systems (19). Specimens NM-O1 and NM-O2 are column-to-footing assemblies connected using a GSS system which uses high strength nonshrink grout on both ends of the sleeve to splice the bars from the footing and column. Specimens LE-O1 and LE-O2 are column-to-pier cap assemblies connected using a GSS system which uses a threaded connection on one end of the sleeve and a grouted connection on the other. Nomenclature for the test specimens is as follows: the first two letters represent the splice sleeve type, GSS with both ends grouted = NM, and GSS with one end threaded and one grouted = LE; the letter “O” stands for original specimen and “R” stands for repaired specimen.

The geometry and reinforcement of the original specimens is shown in Fig. 1. The columns are 8.5 ft tall with a 21 in. wide octagonal cross-section. The column longitudinal reinforcement consists of 6#8 grade 60 bars arranged in a circular pattern. The GSS connectors are located in the footing and pier cap for NM-O1 and LE-O1, respectively, and in the columns for NM-O2 and LE-O2. A #4 grade 60 spiral at a 2.5 in. pitch was provided for transverse column reinforcement. The footing is 6 ft long, 2 ft deep and 3 ft wide; the pier cap is 9 ft long, 2 ft deep and 2 ft wide. The material properties for the precast RC components and the repair are given in Table 1.

Test Assembly and Loading Protocol

The test assembly, shown in Fig. 2(a), applied a lateral load at a point that represents the inflection point of an actual bridge column. The footing and pier cap have spans of 4 ft and 8 ft, respectively. The pier cap specimen was tested upside down, with the pier cap on the strong floor. Loading consisted of a constant axial load equal to

6.0% of the axial load capacity of the column and a displacement controlled cyclic quasi-static lateral load. The lateral load was applied using the loading protocol of Fig. 2(b). Two cycles per drift ratio were used and the amplitude was progressively increased until a minimum 20% drop in the lateral load capacity was reached (20). The drift ratio is the displacement with respect to the distance from the top of the footing or pier cap to the level of the lateral load.

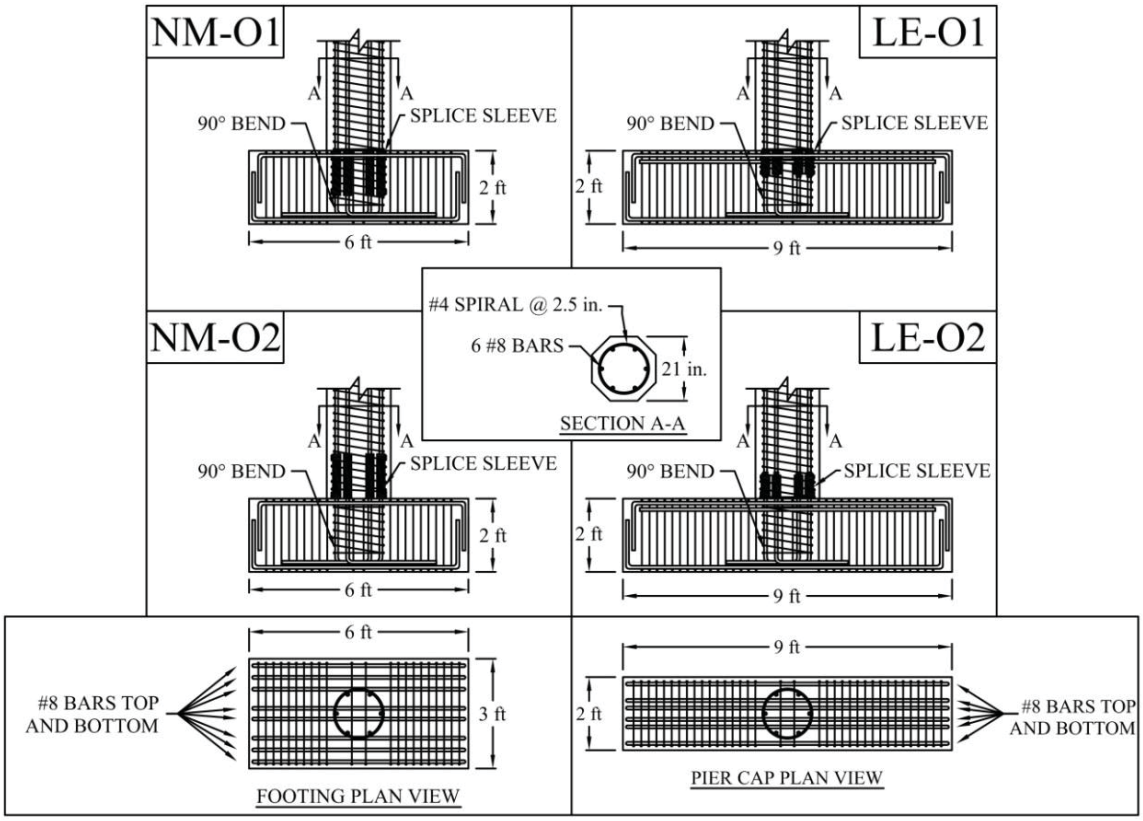
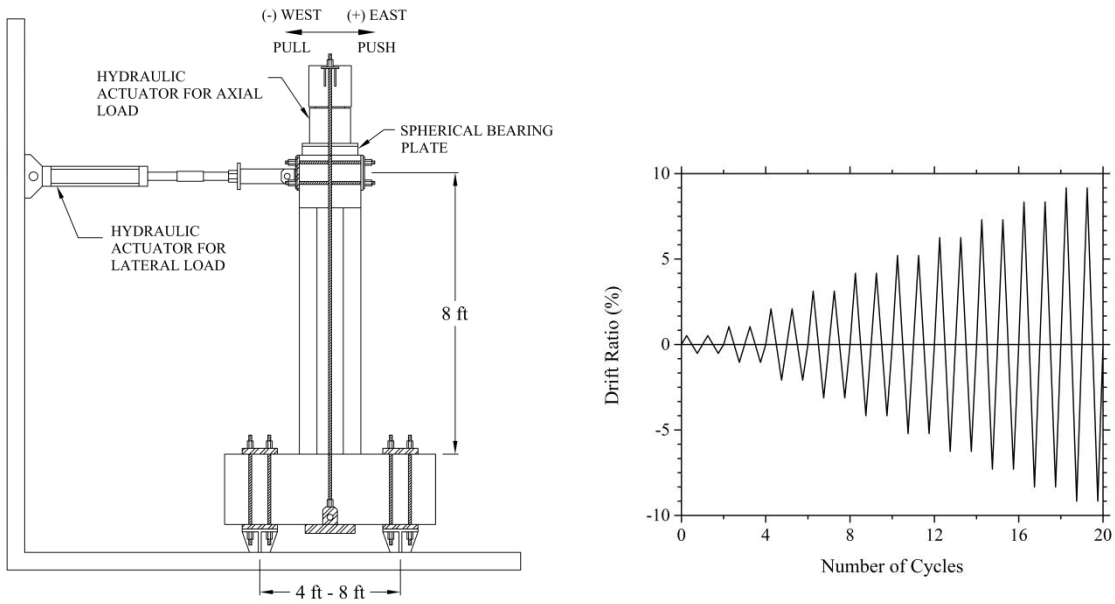


Figure1. Original specimen reinforcement and geometry

Table1. Material Properties

Material Properties		NM-O1	NM-R1	NM-O2	NM-R2	LE-O1	LE-R1	LE-O2	LE-R2
Longitudinal Bars	F_y, ksi (MPa)	68 (469)	68 (469)	68 (469)	68 (469)	68 (469)	68 (469)	75 (517)	75 (517)
	F_u, ksi (MPa)	93 (641)	93 (641)	93 (641)	93 (641)	93 (641)	93 (641)	103 (710)	103 (710)
Concrete Compresive Strength	Test-Day, ksi (MPa)	5.5 (38)	6.4 (44)	8.4 (58)	9.3 (64)	6.0 (41)	6.1 (42)	8.2 (57)	9.4 (65)



(a) (b)

Figure 2. Test Assembly and Loading Protocol: (a) Test Assembly; (b) Loading Protocol.

ORIGINAL TEST SPECIMEN RESULTS

The damage state of the specimens prior to the repair is a critical parameter for the repair design and subsequent performance. The initial test results of NM-O1, NM-O2, LE-O1, and LE-O2 are summarized in Table 2 in terms of maximum lateral load, ultimate drift ratio, displacement ductility, reserve strength, and failure mode. The failure mode of NM-O1, NM-O2 and LE-O1 was fracture of an extreme longitudinal bar, while LE-O2 failed due to multiple longitudinal bars pulling out from the GSS connections in the column. The extreme east longitudinal bar fractured in both NM-O1 and NM-O2. The extreme west longitudinal bar fractured in LE-O1. At failure of the four original specimens the lateral load capacity dropped well below 20% of the ultimate load. The reserve strength of the original columns after testing ranged from 44% to 65% of the lateral load capacity. A very well developed plastic hinge formed at the footing-to-column and column-to-pier cap interface, as shown in Fig. 3. Extensive spalling and cracking occurred in the plastic hinge region of the original specimens. All original specimens experienced flexural cracking which extended up to 14 in. away from the footing or pier cap interface.

To assess the damage state of the original specimens a five-level performance evaluation approach was used (21). This assessment procedure was based on the performance of the structure, which is defined by a particular damage state, and is classified into five levels. Level 1 is equivalent to no damage and level 5 is equivalent to local failure or collapse. According to this type of assessment the four original specimens had reached a damage state designation of level 5, since rebar fracture or pullout from the GSS connectors

occurred, thus significantly compromising the lateral load carrying capacity of the columns. Structural components with a damage level of 5 usually require replacement. However, with the repair method developed in this research, repair of precast columns connected using GSS connectors even at a level 5 damage is possible.

Table 2. Original Specimen Test Results

Test Criteria	NM-O1	NM-O2	LE-O1	LE-O2
Lateral Load, kips (kN)	38.8 (173)	42.0 (187)	36.3 (161)	44.8 (199)
Ultimate Drift Ratio, %	6.69	7.91	6.50	6.00
Displacement Ductility	6.1	6.8	5.8	3.1
Reserve Strength, kips (kN)	21.4 (95)	23.6 (105)	20.6 (92)	15.9 (71)
Failure Mode	East Bar Fracture	East Bar Fracture	West Bar Fracture	Bar Pullout

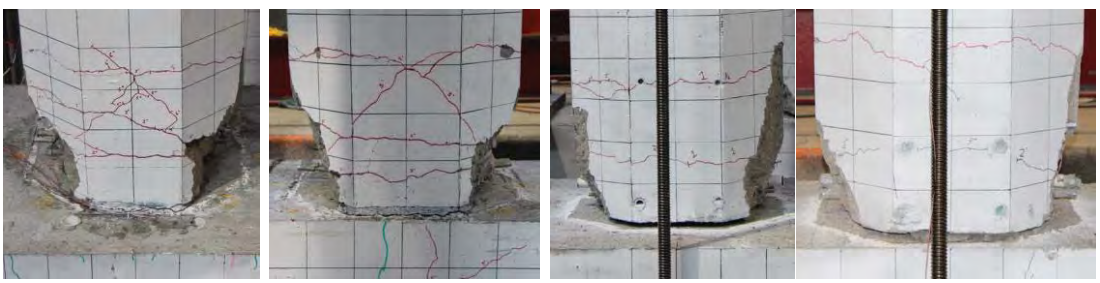


Figure 3. Original Specimen Damage: (a) NM-O1; (b) LE-O1; (c) NM-O2; (d) LE-O2.

REPAIR DESIGN

The objective of the repair was to strengthen the original plastic hinge region by increasing the cross-section from a 21 in. octagonal section to a 30 in. diameter circular section. The 30 in. diameter circular cross-section was constructed by post-installing epoxy anchored headed bars for additional tensile reinforcement and then filling a CFRP shell with nonshrink or expansive concrete, as shown in Fig. 4. To form the new plastic hinge, a bending moment referred to as MM must be developed at the desired plastic hinge location. In the present case, the original specimen test results were used to determine M_{PH}; however, this bending moment can also be found using sectional analysis. From Fig. 5, the bending moment demand experienced at the column joint, MM_{joint}, is a function of the length of the repair, HH_{rep}, and the distance from the theoretical point of inflection to the column-footing or column-pier cap joint, HH_{col}. This relationship is shown in Eq. (1).

$$M_{joint} = \frac{M_{PH}}{\left(1 - \frac{H_{rep}}{H_{col}}\right)} \quad [1]$$

Similar to the bending moment demand, the shear force demand that must be resisted by the column to achieve plastic hinge relocation, VV_{PHR}, is directly related to HH_{rep}. This relationship is shown in Eq. (2).

$$V_{PHR} = \frac{M_{PH}}{(H_{col} - H_{rep})} \quad [2]$$

Eqs. (1) and (2) indicate that using the minimum possible repair height is advantageous for limiting the bending moment and shear demands. However, the height of the repair must be sufficient to relocate the new plastic hinge to a column cross-section which has only minor damage. From the observed damage of the four original specimens shown in Fig. 3, a repair height of 18 in. was determined to be sufficient. In this case there were two criteria to define the repair height. The first criterion was to relocate the plastic hinge above any structural cracks equal to or larger than 0.01 in. wide. The second was to provide enough height to develop the headed bars in tension.

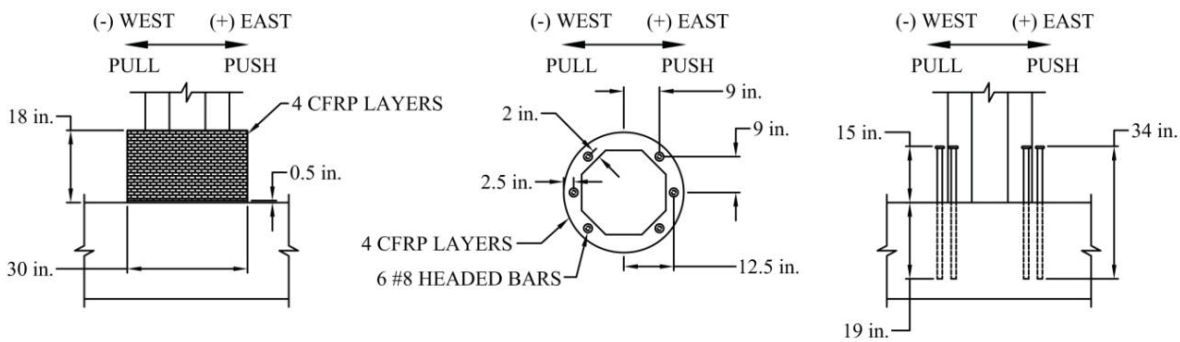


Figure 4. Repair design.

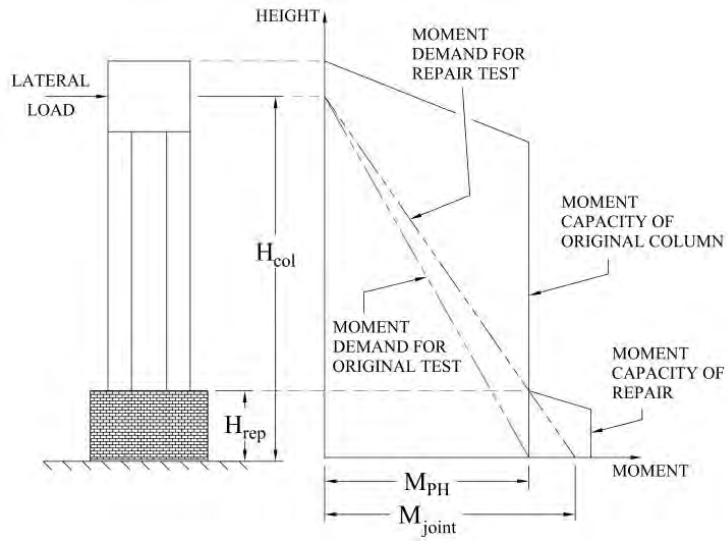


Figure 5. Bending moment demand.

Headed bars were designed to develop the increased joint bending moment, MMjoint, required for the repair.

The headed bar length drilled into the footing or pier cap was determined so that the epoxy anchorage would develop the bars in tension. Similarly, the length of headed bar extending into the repair was checked for adequate development length. These parameters led to the design of 6#8 grade 60 headed bars which were post installed around the column as shown in Fig. 4. Note that this is the same cross-sectional area of longitudinal reinforcement used in the original column. The embedment into the footing or pier cap was 19 bar diameters and the length extending into the repair was 15 bar diameters. The headed bars used in this design had a head diameter of 2.25 in. and yield strength of 62 ksi.

The 30 in. diameter cross-section of the repair utilized a CFRP shell which was designed to provide concrete confinement, shear strength, and was also utilized as stay-in-place formwork for the nonshrink or expansive concrete.

A total of four layers of unidirectional CFRP sheets oriented in the hoop direction were provided; one layer was provided to restore the shear strength of the original plastic hinge region, and three layers were provided for confinement and prevention of strain softening; details of the design procedure are provided elsewhere (22-24). A 0.5in. gap was left between the bottom of the jacket and footing or pier cap surface, as shown in Fig. 4, to ensure there was no bearing of the CFRP shell on the concrete during large displacements. The ultimate tensile capacity of the CFRP composite was 101 ksi [696 MPa], the modulus of elasticity was 8990 ksi [62000 MPa], and the ultimate strain was 1.12%, as determined by tensile coupon tests according to ASTM D3039 (25).

The shear capacity of the original column should be checked to ensure ductile flexural failure at the location of the relocated plastic hinge. In this case, the transverse reinforcement in the relocated plastic hinge was sufficient to produce a flexural failure mode. If however, the shear capacity of the column were insufficient, additional retrofit of the column would be necessary.

Repair Procedure

Before beginning the repair the damaged columns were re-plummed. Prefabricated CFRP shells were created by wrapping and curing a single 18 in.-wide CFRP layer around a 30 in. diameter sonotube to create the proper shape. While the CFRP shell was curing, the holes for the headed bars were core-drilled into the footing or pier cap and the headed bars were epoxy anchored in-place, as shown in Fig. 6(a). After the CFRP shell had cured, it was cut in two half-cylinders and brought around the column as shown in Fig. 6(b). The sonotube inside the shell was used to ensure that the shell maintained its shape, while the additional layers of CFRP were applied and was subsequently removed once all CFRP layers had cured. A 12 in. long by 18 in. wide piece of CFRP sheet was used to splice the CFRP shell halves on both sides. Once the first layer of the CFRP shell was spliced, three additional

CFRP layers were added as shown in Fig. 6(c). Each layer was 100 in. long by 18 in. wide, providing 6 in. overlap for each layer.

This was the last step in completing the construction of the CFRP shell which acted as stay-in-place formwork for the repair concrete. Once the CFRP shell had cured for 7 days, nonshrink or expansive concrete was added between the column and CFRP shell as shown in Fig. 6(d). The repair concrete had a compressive strength of 7.0 ksi.

(a) (b) (c) (d)

Figure 6. Repair procedure: (a) post installed headed bars; (b) split CFRP shell; (c) CFRP shell around column; (d) CFRP shell filled with nonshrink or expansive concrete.

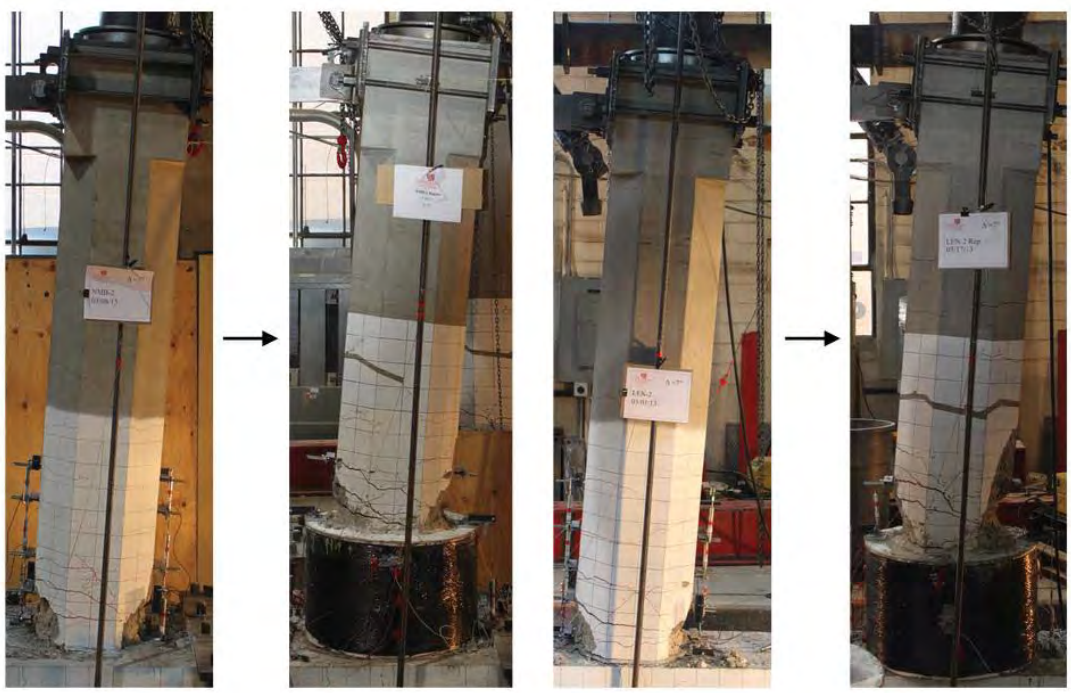
For LE-O1 and LE-O2, the diameter of the repair was larger than the width of the pier cap. Wooden forms were placed alongside the pier cap to provide sufficient width for the repair as shown in Figs. 6(b) and 6(c). The wooden forms were removed once the concrete had cured. In practice, the pier cap would

be oriented above the column and the gap between the repair and pier cap would provide an inlet for the concrete and the gap between the column and the repair would need to be sealed.

EXPERIMENTAL RESULTS FOR REPAIRED SPECIMENS

Since the damage state of all original specimens was similar, the same repair design was used for all specimens. The repair procedure was implemented for NM-O1, NM-O2, LE-O1 and LE-O2 and the repaired specimens are referred to as NM-R1, NM-R2, LE-R1 and LE-R2, respectively, where "R" stands for repaired specimen. The only difference in the repair was the type of concrete used to fill the void between the original column and CFRP shell. This concrete, referred to as the repair concrete, was nonshrink concrete for NM-R1 and LE-R1, and expansive concrete for NM-R2 and LE-R2. The use of expansive instead of nonshrink concrete converts the confinement provided by the CFRP shell from passive to active by pre-tensioning the CFRP shell.

The difference in active and passive confinement among the repaired specimens can be seen by the amount of pre-tensioning that was experienced by the CFRP wrap prior to testing. Strain gauges were used to monitor pre-tensioning for all repaired specimens. Specimens NM-R1 and LE-R1 designed with nonshrink concrete had low pre-tensioning between 0.016% and 0.015%, while specimens NM-R2 and LE-R2 designed with expansive concrete had significant pre-tensioning between 0.150% and 0.180%.



(a) (b) (c) (d)

Figure 7. Plastic hinge relocation: (a) NM-01; (b) NM-R1; (c) LE-01; (d) LE-R1.

The test assembly and loading protocol were the same for the original and repaired specimens. The successful plastic hinge relocation of NM-R1 and LE-R1 is shown in Fig. 7. The strength and displacement capacity of the damaged bridge columns was restored for all repaired specimens.

Specimen NM-R1

The hysteretic response of NM-R1 superimposed with the hysteretic response of NM-O1 is shown in Fig. 8(a). From Fig. 8(a) and Table 3 it is clear that NM-R1 achieved a lateral load 18% higher than NM-O1 and had a similar displacement capacity. The failure mode of NM-R1 was fracture of column longitudinal bars in the relocated plastic hinge region. The extreme west longitudinal bar fractured during the first cycle of the 7.3% drift ratio and the extreme east longitudinal bar fractured during the second cycle of the same drift ratio. The east longitudinal bar fractured only 21.5 in. above the original fracture location in the original specimen NM-O1. This implies that the repair provided sufficient confinement and clamping force to develop the longitudinal bar in a short distance. Other major events included onset of significant spalling at a 3.1% drift ratio, and CFRP circumferential cracking in the fiber direction at a drift ratio of 4.2%. The CFRP crack was located approximately 3 in. below the top of the repair, at the same level as the top of the headed bars, and extended halfway around the jacket circumference on the east side.

This is the same side the longitudinal column bar had fractured in NM-O1. The hysteretic response of the specimen was unaffected by the circumferential crack in the CFRP shell.

Specimen NM-R2

The hysteretic response of NM-R2 superimposed with that of NM-O2 is shown in Fig. 8(b). The failure mode of NM-R2 was fracture of the extreme west longitudinal bar during the 5.2% drift ratio. The lateral load capacity of NM-R2 was 28% higher than the lateral load capacity of NM-O2, as shown in Table 3. However, the displacement capacity of NM-R2 was less than that of NM-O2, at the ultimate displacement defined by a 20% drop in lateral load.

The longitudinal column bar fracture, which caused the 20% drop in lateral load, was due to embrittlement from a tack weld to hold instrumentation fixtures to the bar. The brittle fracture of the bar was obvious through several characteristics of the fracture. First, the fracture location was 10.5 in. above the top of the repair, which is significantly higher than the fracture location of all other tests which occurred within 5 in. of the column-repair interface; second, the fracture plane of the bar was smooth and level which is a characteristic of a brittle steel fracture plane; third, there was no decrease in diameter of the fractured bar when compared to the original diameter indicating no necking prior to the fracture.

Although a 20% drop in lateral load carrying capacity was observed, the test was carried out through the

8.3% drift ratio. The hysteretic response shows that despite the welding mishap, NM-R2 performed quite well in the west direction of testing after the column bar had fractured, outperforming NM-O2.

Specimen LE-R1

In the case of specimen LE-R1, a monotonic pushover was performed in addition to the loading protocol of Fig. 2(b). The monotonic load was applied to the column in the east direction up to a drift ratio of 6.9%. At this point, the column was brought back to its original vertical position and tested according to the loading protocol of Fig. 2(b).

This series of loading emulates a near fault ground motion which is characterized by an acceleration pulse followed by sinusoidal ground motion.

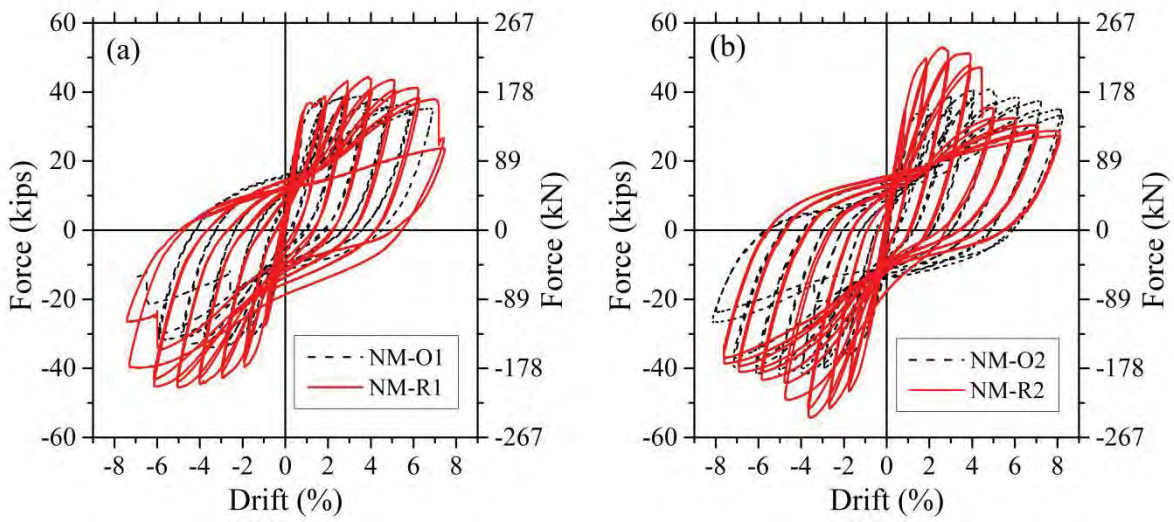


Figure 8. NM Hysteretic Response; (a) NM-R1 & NM-O1; (b) NM-R2 & NM-O2

Table 3. Repaired Specimen Test Results

Test Criteria	NM-R1	NM-R2	LE-R1 (Pushover)	LE-R1 (Cyclic)	LE-R2
Max Lateral Load, kips (kN)	45.6 (203)	53.6 (238)	46.8 (208)	40.5 (180)	50.5 (225)
Ultimate Drift Ratio, %	6.96	4.83	6.88	7.2	6.17
Displacement Ductility	6.0	3.7	6.6	---	4.6
Failure Mode	West & East Bar Fracture	West Bar Fracture	---	East Bar Fracture	CFRP Wrap Fracture

The monotonic pushover curve is shown in Fig. 9(a). Although the column was displaced to a drift ratio beyond the ultimate drift ratio of LE-O1, no longitudinal bars fractured in the column due to the

monotonic nature of the load. There was major spalling on the east side of the column, as shown in Fig. 7(d), which extended 20 in. up the column and exposed the spiral reinforcement.

With the repaired column already damaged in one direction from the monotonic pushover test, the specimen was subsequently tested cyclically. The hysteretic response of LE-R1 is also shown in Fig. 9(a) superimposed with that of LE-O1. The right side of the hysteresis for LE-R1 shows an irregular response due to damage from the monotonic pushover. The left side of the hysteresis seems to be minimally affected; comparisons of the hysteretic response are made to this side of the hysteresis. The failure mode of LE-R1 was facture of the extreme east longitudinal bar in the relocated plastic hinge region. The bar fractured during the first cycle of the 7.3% drift ratio.

Similar to the behavior of NM-R1, the onset of significant spalling on the west side of the column occurred at a 3.1% drift ratio and the onset of circumferential CFRP cracking occurred at a 4.2% drift ratio. Cracking was located approximately 3 in. below the top of the repair, at the top of the headed bars, and extended half way around the

CFRP jacket circumference on the west side; this crack occurred on the same side as the longitudinal bar fracture in LE-O1. The specimen remained unaffected from the crack in the CFRP shell.

Due to the initial damage of LE-R1 from the monotonic pushover it is difficult to directly compare LE-R1 to LE-O1. However, by examining the performance of LE-R1 in Table 3 from both the monotonic pushover and cyclic tests, it is clear that LE-R1 performed similarly to LE-O1.

Specimen LE-R2

The hysteretic response of LE-R2 superimposed with the hysteretic response of LE-O2 is shown in Fig. 9(b).

During the 3.1% drift ratio, a crack occurred and extended over the entire circumference of the CFRP shell, which correlated with the top of the headed bars. Failure of LE-O2 was due to pullout of the longitudinal column bars after yielding on both column sides; this caused additional demand on both sides of the repair causing the circumferential crack in the CFRP shell on both sides.

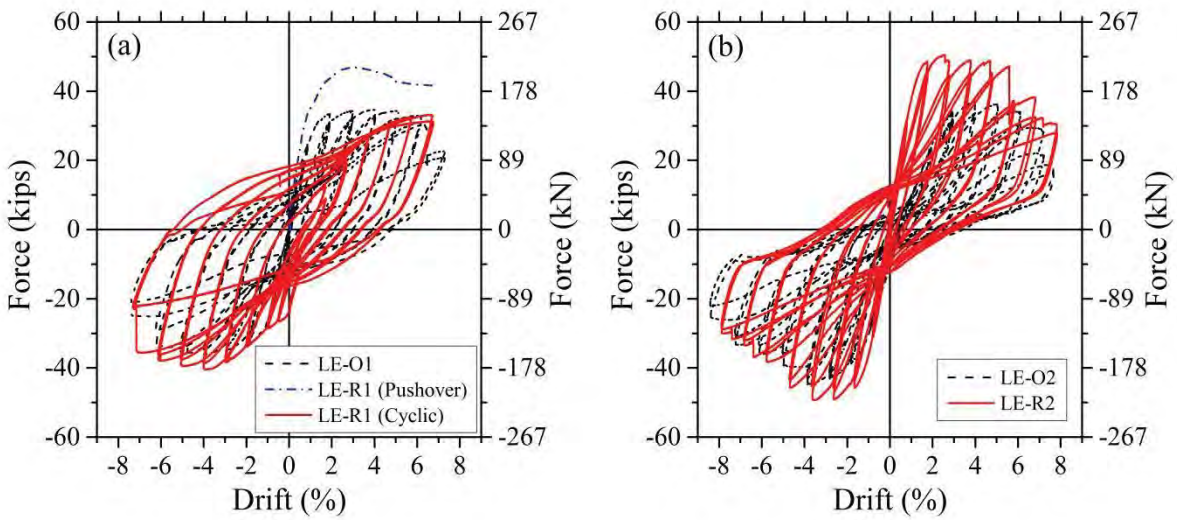


Figure 9. LE Hysteretic Response; (a) LE-R1 & LE-O1; (b) LE-R2 & LE-O2.

Before the plastic hinge was completely relocated above the repair, the CFRP shell fractured. Fracture of the CFRP shell occurred during the first cycle of the 6.3% drift ratio, which caused a 20% drop in the lateral load. This fracture occurred directly below the top of the headed bars and the circumferential CFRP crack on the north-east side of the repair. Although a 20% drop in lateral load carrying capacity was observed during the 6.3% drift ratio, the test was continued through the 8.3% drift ratio. As the test progressed, the CFRP jacket fractured three additional times, with each fracture moving closer to the column pier cap interface.

Despite the fact that the plastic hinge was not relocated entirely above the repaired region, the specimen still showed acceptable hysteretic performance. The lateral load capacity of LE-R2 was 13% higher than the lateral load capacity of LE-O2. However, once the CFRP jacket had fractured, the hysteretic response of LE-R2 followed closely the response of LE-O2.

The reasons for failure of LE-R2 in the CFRP shell rather than in the column cross-section adjacent to the repair are: (i) the GSS connectors were located in the column leading to a different failure mode of the original specimen, which is pullout failure of the GSS rather than rebar fracture; as such, the plastic hinge in LE-O2 is shorter than when the sleeves were located in the pier cap as in LE-O1. With a shorter plastic hinge, damage does not spread up the column, implying that the repair could have been shorter, thus reducing the flexural demand in the repaired region; (ii) the strength of the column cross-section adjacent to the repair; by comparing material properties between LE-O1 and LE-O2 the yield strength of the longitudinal bars was 10% higher and the concrete compressive strength was 54% higher. The stronger column cross-section combined with minimal damage of the original specimen increased the required

moment capacity to higher levels than expected, thus causing failure to occur. Both reasons relate to the original damage state of the column. Therefore, the importance of having a good assessment of the damaged column strength cannot be overstated.

CONCLUSIONS

A repair procedure for post-earthquake damage has been developed for severely damaged precast bridge columns connected using GSS connectors located in the column, footing, or pier cap. The repair converts the original plastic hinge region of an octagonal column to a larger circular cross section, thereby relocating the new plastic hinge to a section adjacent to the repair. This repair procedure was implemented for damaged precast bridge column-to footing and column-to-pier cap assemblies which were tested under quasi-static cyclic loads.

The repair was capable of restoring the performance of the specimens in terms of lateral displacement, lateral load, energy dissipation and stiffness.

The important components of the repair method were a CFRP shell, post-installed headed steel bars and concrete inside the CFRP shell. The CFRP shell provided confinement, shear strength and hoop tension. The postinstalled headed bars were successful in providing sufficient flexural capacity to relocate the plastic hinge; they provided a means to transfer the tension lost by the fractured original column longitudinal bars connecting the columns to the footing or pier cap.

Both nonshrink and expansive concrete were successful in restoring the capacity of the column. The nonshrink concrete in the CFRP shell provided sufficient passive confinement. The expansive concrete in the CFRP shell provided active confinement. The use of expansive instead of nonshrink concrete caused sufficient expansion to produce an active confinement system. However, control of the amount of concrete expansion is important as excessive initial expansion will reduce the remaining tensile capacity of the CFRP shell.

Based on the overall performance of the repaired specimens this is a viable technique for damaged precast RC columns in seismic regions. In the present case, initial damage of the precast RC columns was severe therefore the repair method is robust and would be applicable to precast RC columns with varying damage states. The repair technique can be installed in eight days which is advantageous in emergency response situations and is an excellent application for accelerated bridge repairs.

ACKNOWLEDGMENTS

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NOTATION

H_{col}: distance from the theoretical point of inflection to the column-footing or column-pier cap joint

H_{rep}: length of the repair

M_{joint}: bending moment demand experienced at the column joint

M_{PH}: bending moment to form new plastic hinge

V_{PHR}: shear force demand that must be resisted by the column to achieve plastic hinge relocation

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THE SEISMIC DUCTILITY DEVICE: A NEW CONNECTION FOR PRECAST CONCRETE COLUMNS DESIGNED FOR SEISMICALLY ACTIVE AREAS

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ABSTRACT

Seismic Ductility Device (SDD) is a novel structural component providing ductility at the plastic hinge regions of columns using Shape Memory Alloy (SMA) rods and a core consisting of Engineered Cementitious Composite (ECC) and a steel shear pin.

The SDD is designed to be prefabricated as a modular element and placed between the column and pier cap at the expected plastic hinge. This allows faster bridge construction and lower costs by enabling precast—instead of cast-in-place—columns. Employing an integrated set of tested technologies, the SCS Rapid Column System SDD provides an efficient and cost effective solution to achieve seismic performance with easy inspection, maintenance and repair. A non-seismic version is also available.

INTRODUCTION

Precast columns are the most difficult component to properly connect in Accelerated Bridge Construction, especially for seismic ductility. Most existing solutions are versions of cast-in-place configurations of rebar and grout which reproduce the cast in place concrete methods of seismic survivability. The Seismic Ductility Device (SDD) developed by Structural Component Systems, Inc. is based on research using innovative materials to provide stiffness reduction and hysteresis in the plastic hinge region of the column. The SDD provides an easy to construct, inspectable and easily field repairable hinge connector element.

Existing solutions for seismic connections at column plastic hinges involve considerable damage to the column during an event. Because the intent of design codes is to prevent collapse and not necessarily maintain operation, these solutions allow the bridge to survive but may take considerable time and expense to repair. In some cases, the structure may be a total economic loss after an event despite meeting the code-intended survivability.

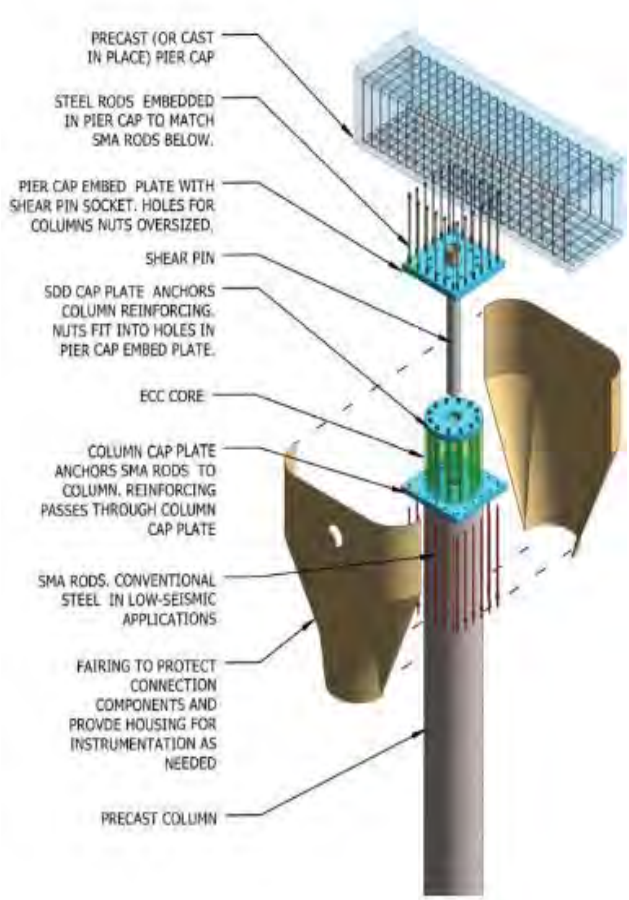

Figure 1. Exploded View: Column with Biaxial SDD. Patent Pending.

The SDD is essentially a structural fuse that is intended to dissipate seismic energy and also "tune" the structure to a longer period, which, in most cases, means lower realized seismic forces in the structure. The SDD is designed to be easy to assemble in the field and easy to inspect and repair after a seismic event.

The key materials used in the SDD are Shape Memory Alloy (SMA) rods to carry tension forces from moments and Engineered Cementitious Composite (ECC) concrete to carry compression. These materials have very good ductile performance without incurring permanent set or significant damage under most seismic design scenarios. The SDD also has unique geometry that allows the shear, moment tension and compression to be carried by separate elements to optimize their performance.

The SDD's most unusual characteristic is that the SMA rods are *outside* the ECC core of the column. This geometry allows easy field assembly, inspection and repair. The SMA elements and ECC core are enclosed in a non-structural hollow fairing which allows opportunities for instrumentation housing, lighting and signage.

DEVELOPMENT

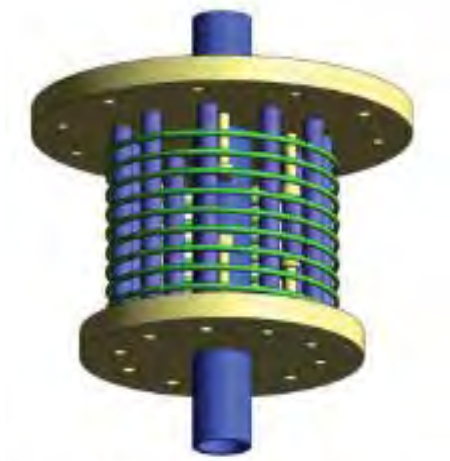

Figure 2. The UNR SDD Assembly

The SDD design is based on extensive testing of Engineered Cementitious Composite(ECC) concrete and Shape Memory Alloy (SMA) materials at the University of Nevada Reno(UNR).These materials as developed into the SDD provide ductility at plastic hing locations in the column. The UNR version, shown in Figures 2and,3 separates the key components of force (compression, shear and moment) into different parts of the SDD to provide the most reliable and predictable ductile response. The materials that make the SDD possible are SMA rods and an ECC core. Tazarv and Saiidi (1) report analytical and scale dynamic test results that show the combination of SMA and ECC materials provide reasonable energy absorption and excellent damage tolerance in concrete columns.

The SMA rods are fabricated to have “super-elastic” properties that allow the rod to stretch significantly (6% strain) but with minimal permanent set. The ECC used in the core has internal tension capacity that results in compression performance as if the material has additional confinement.

Subsequent development at UNR involved a “de-constructible “structural connection element that combined the ECC, SMA and shear pin concepts that had been tested separately over the years. The UNR testing program has demonstrated excellent performance when subjected to seismic loads in the lab. Varela (2) (3) reports that preliminary results for the UNR component indicated it performed well with little cracking in the core and some broken SMA rods at extreme dynamic levels. The rods were replaced, the cracks repaired and performance of the SDD was restored as demonstrated in additional testing. The UNR SDD is an exciting system that addressed modularity and provided excellent seismic performance.

Based on the UNR testing, subsequent improvements have been developed by SCS to provide straightforward constructability and easy inspection and repair. In the UNR SDD, the SMA bars could not be easily inspected or replaced. With the SMA bars deep

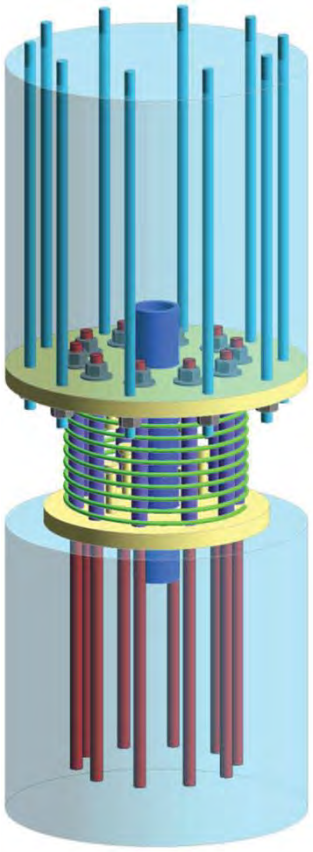

Figure 3. The UNR SDD within the Column. The column is above the SDD in this configuration. The upper rods are the column reinforcing. The lower bars are SMA.

inside the ECC core, there was no way to determine their condition pre- or post- earthquake. To replace the rods (there isn't any repair method available) the SDD had to be removed from the column. Thus either the column has to be lifted off the SDD (if at the bottom) or the pier cap has to be lifted off the SDD. Such access is not conducive to easy inspection or repair.

The key aspect for the SDD is the placement of the SMA rods outside the column and core. In this location they can be readily inspectedand replaced without taking the bridge out of service. The core design essentially matches the design of the column below, so there is no change in elastic capacity for axial force and moment below the SDD. Based on review of the UNR testing, the most likely damage to a SDD is to the SMA rods. Cracks may occurs in the ECC core and thesecan be repaired by injection from the sides without removing the SDD from the bridge. The SMArods are used to their best effect for tension loads and are isolated from compression loads. This designalso allows the use of SMA cables in lieu of SMA rods in some applications.

Obviously the downside to moving the rods outside the column is that the SMA and portions of the plates they connect would be exposed to the environment. One of the guiding principles in concrete structures of all types is to bury steel within the concrete for environmental protection. The SMA Nitinol material is highly corrosion-resistant (one of the major commercial applications for Nitinol is orthodontia and medical implants). However, the steel plates would still be susceptible to corrosion as would nuts and other hardware. Using concrete cover is not the only way to protect metal components. The SCS SDD system uses a fairing cover made of fiberglass or GFRC to protect the SDD connection. This fairing seals to the column and the girder above or the foundation below if the SDD is at the bottom of the column. This solution offers several advantages:

- The architectural appearance can be separate from the concrete structural members. Current practice is to use custom formwork to pour in place columns with flares or other features (flared column tops are very popular). The custom formwork is very expensive and the odd shapes of the flare columns can lead to seismic performance issues - see Nada et al (4). As the fairing does not support bridge loads it can be of any configuration that will protect the connection from the environment.
- The fairing allows inspection of the rods and core through built-in ports. The fairing can be removed to provide complete access as needed for repair.
- The fairing has space for instrumentation for bridge monitoring. Conduits connecting the SDD space to the girder and column can be placed within the girder and/or column respectively. This allows internal routing of instrumentation wiring.

- The fairing can host signage or lighting with wiring accessible inside the fairing.

DESIGN APPROACH AND METHODOLOGY

The SDD allows a precast column design with connections at the top and bottom of the column. Each connection can be either ordinary moment (CCD), seismic moment (SDD) or pin connection. The connections consist of embed plates in the foundation and pier cap that provide for a formed socket for the shear pin and a number of threaded couplers to connect the SMA or steel connections to the embedded steel anchor rods. In each connection there is a grout or ECC core that provides vertical compression load resistance. In the case of a pin connection, only the shear pin is used.

The design of the column is per AASHTO LFRD standards. The vertical reinforcing is either threaded at the ends or special threaded bar is used to allow the column cap plate or SDD cap plate to be installed.

The current product charts use 5000 psi normal weight concrete as the column design basis. Columns are pre-designed using 1%, 2% and 3%, 60 ksi vertical steel ratios. The column would still have the required seismic ties or spirals per the AASHTO code.

The design of the shear pin is based on limitations of the shear capacity of the column. In this way the shear capacities are matched and there are no weak links for shear. The pin sits on the bottom of the lower socket and the upper socket has a clearance to keep vertical load from the shear pin.

The design of embeds is based on the standard anchorage of the rods using either an approved anchorage device or typical bond length. Design of steel components for anchorage (rods and nuts) or plates are designed per AASHTO standards.

The design of the SDD separates the three key force components at the plastic hinge (shear, tension and compression) into specific materials to allow the device to have predictable and adequate performance.

Shear forces at the plastic hinge are carried by a steel pin on the interior of the device. The pin does not bear on or connect to the top of the socket; therefore the pin cannot carry compression or tension. This decouples most shear forces from the ECC core. The use of the SMA and ECC require a few adjustments to the standard concrete methods available in AASHTO for design.

SDD Design

The design of the SDD uses the same principles as AASHTO LFRD reinforced concrete design. The design section is the intersection of the pier cap or foundation embeds plate and the column top plate. Prior to non- linear performance, strains are assumed to be linear but the strain distribution is based on the ECC core strain of 0.004 which is a more realistic value of strain near crushing. Yield strain for the Nitinol components is taken as 0.01 (F), approximately 5x larger than steel yield strain of 0.00207. The columns steel rebar is continuous through and anchored to the top of the ECC core. Thus the reinforcing in the core is available for compression only. Because the SDD plastic hinge is evaluated at the interface between the pier cap and the SDD top plate, reinforcing steel is not available for tension loads as it is not continuous into the pier cap.

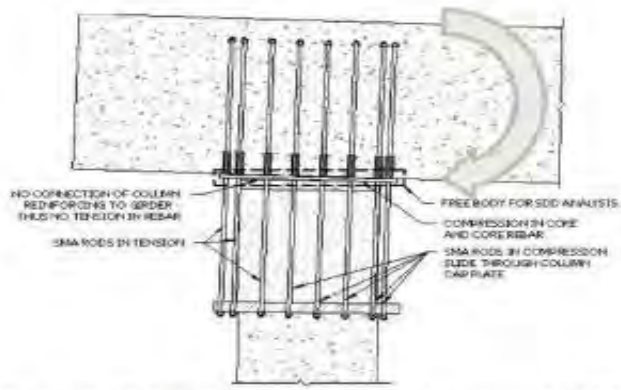

Figure 4. SDD Under Rotation. The pier cap embed lifts off the SDD cap plate and engages the SMA rods on the tension side.

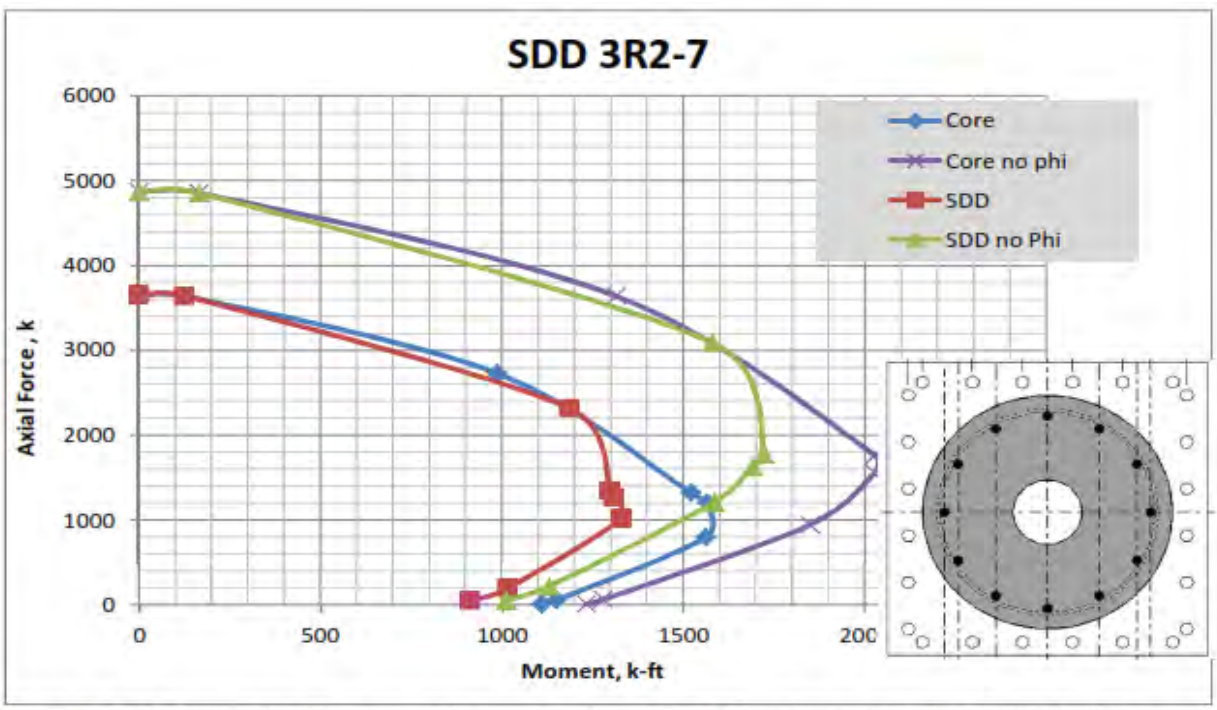

Figure 5. 3R2-7 Interaction Diagram and Cross Section. The SMA rods are indicated by the hex bolt heads; the dots are the vertical column rebar. This design is for a 36" diameter concrete column with 2% vertical steel. The SMA rods are 7/8" diameter.

The strength limit curve shows that axial performance at minimal moment is the same as the ordinary column. In fact the SDD limit matches the core very closely until near the balanced condition. The same

phi factor is used for both the column/core and the SDD/SMA. This is technically code compliant but the transition for phi is based on the strain of steel tension rebar, which is not truly applicable to the SDD's SMA materials. The corresponding curves without any phi factor are shown for comparison. Note that the SDD will always yield before the core/column. In this model, the column/core has (12) #11 bars for 2% steel area. There are (24) #7 SMA rods distributed uniformly around the compact square SDD baseplate as shown. The compression-only portions of the curves are identical as the SMA components are inactive.Because the modulus of the Nitinol is low, the strain associated with the SDD is larger for any given moment that develops tension in the SMA bars. Thus the rotational stiffness is considerably less once the bars are placed in tension. For example, the rotational Keff (linear stiffness to first yield of the SMA rods) for the 3R2-7 product shown above is approximately 139.5 X 103 k-ft/rad. For comparison, the section rotational Keff for the column is 979.6 X103 K-ft/rad (neglecting the length of the column.) The lower rotational stiffness of the SDD should, in most bridge geometries and locations, reduce seismic demand by lengthening the period of the structure. The SDD, unlike other isolation-type systems, does not affect the vertical stiffness or strength of the columns assembly because it only "activates" once sufficient moment has been generated to place the SMA rods in tension.The SDD is only one portion of a bridge structural system – the SDD stiffness and performance has to be incorporated into the engineering analysis of the entire bridge geometry.

SDD NON-LINEAR PERFORMANCE

Nitinol SMA

The SMA Nitinol material provides the majority of the hysteretic behavior of the SDD. An idealized Nitinol hysteresis diagram is shown in Figure 6. Note the "flag-shaped" curve typical of this material. The initial stiffness (k_1) is based on the austenite form of the Ni-Ti crystal structure. The post-yield stiffness (k_2) reflects the transition to the martensite crystal structure. Once the load is reversed, the initial recovery stiffness is k_1 until the "lower plateau" stress is reached (65% of f_y). Once the lower plateau is reached, the stiffness returns to k_2 until it intersects the original elastic curve where the stiffness returns to k_1. The typical limit for minimizing residual strain is a maximum applied strain of 6%. Beyond the 6% limit, the stiffness is 30% of k_1 and there will most likely be residual strain in the material.

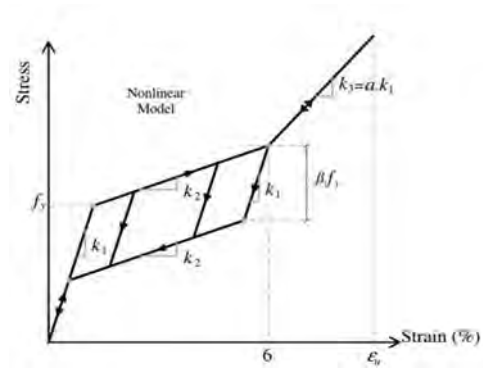

Figure 6. Nitinol Hysteretic Model from Tazarv and Saiidi(1)

The model shown is the result of testing and analytical work at the University of Nevada Reno as reported in Tazarv and Saiidi (1). Based on this work, the report presents the table below for suggested design parameters for Nitinol reinforcing bars in bridge columns:

Table 6.2- Minimum and Expected Tensile NiTi SE SMA Mechanical Properties

Parameter	Minimum*	Expected**
Austenite modulus, k_1	4500 *ksi* (31025 *MPa*)	5500 *ksi* (37900 *MPa*)
Post yield stiffness, k_2	--	250 *ksi* (1725 *MPa*)
Austenite yield strength, f_y	45 *ksi* (310 *MPa*)	55 *ksi* (380 *MPa*)
Lower plateau stress factor, β	0.45	0.65
Recoverable superelastic strain, ε_r	6%	6%
Secondary post-yield stiffness ratio, α	--	0.3
Ultimate strain, ε_u	10%	10%

* To be used in material production

** To be used in seismic design of SMA-reinforced concrete members

Figure 7. Nitinol Recommended Design Values from Tazarv and Saiidi(1)

SCS uses the "expected" values from this table for the analytical design of the SDD.In the SCS SDD de sign, the SMA rods and SDD core are set such that the height of the core is at least 10X the diameter of the SMA rods. This allows a "stretch length" which has been shown to be helpful in mild steel rods undergoing large strains. In addition, the extra length allows slight fit up adjustments during assembly. The illustrations in this report exaggerate the core height for clarity.

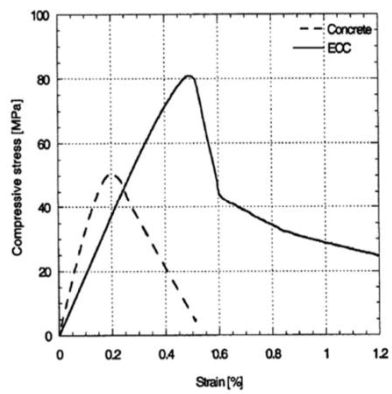

Figure 8. Compressive Stress-Strain Curve for ECC. After Fischer and Li (4).

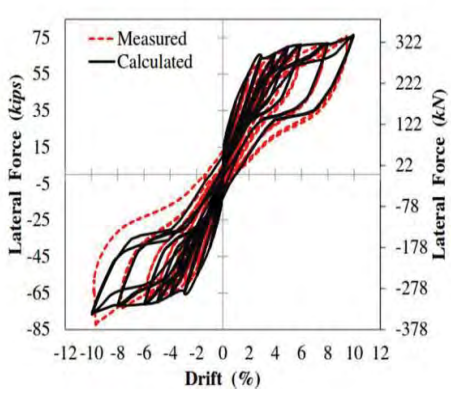

Figure 9. Typical UNR SMA/ECC Hysteretic Performance. Note the flag shaped curve characteristic of the SMA material with little additional hysteresis signature from the ECC. Reproduced from Tazarv and Saiidi (1)

ECC core

ECC is often noted for its direct tension capacity. But in the SDD, the most important aspects of the ECC are essentially self-confinement and strain capacity. The small polyvinyl alcohol fibers that give ECC its tensile properties also improve compressive

strength and strain capability by preventing micro-cracks from linking into failure cracks. This leads to a more gradual compressive failure than ordinary concrete and substantial additional deformation is available while still carrying compressive load. Figure 8 illustrates typical ECC compresion stress-strain performance.

In evaluating test results, Tazarv and Saiidi (1) suggest that the tensile performance of the ECC be neglected in computer analysis of the ECC column systems. In the case of the SCS SDD, the design concept is based on the concept that tensile loads on the ECC should never be significant.

In the SDD application, the ECC core is intended to remain uncrushed under most cyclic loads. Thus energy disappation is mostly from the SMA material properties; the ECC, due to its more deformation-friendly nature, allows the larger displacements without damage. This concept is borne out by reviewing a typical UNR result shown in the figure. Note that there is little hysteresis beyond the “flag shape” typical of the SMA material. This was also evident in review of damage to the UNR SDD after testing: some SMA rods were damaged or broken. The ECC damage consisted of a few cracks that could have been repaired in-place in an SCS SDD design.

By using an ECC material in the core, the strains expected in the SMA material can be accommodated with little if any damage to the core. This preserves vertical load capacity after an event even if several SMA rods are damaged or broken. Although the core ECC will be subjected to micro-cracking, the integrity of the core will be maintained for many cycles.

NON-SEISMIC APPLICATION –THE COLUMN CONNECTION DEVICE (CCD)

The SDD design is rooted in the search for better ductility devices for bridge construction. The SCS SDD is a result of thinking about how to incorporate modular construction, inspection and maintenance in the SDD concept. If, however, the ductility of an SDD is not required, the remainder of the system can be

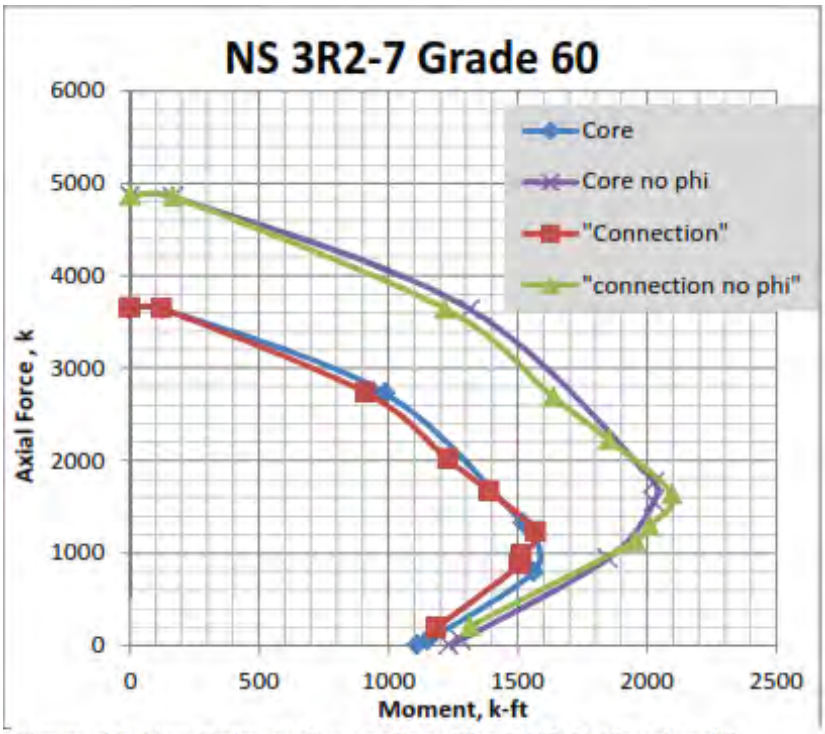

Figure 10. Non-Seismic Connection: The SCS CCD using #7 Grade 60 Rebar

adapted to provide the modular construction, ease of inspection and maintenance benefits for bridge construction. This non-seismic version is called the Column Connection Device (CCD).

The geometry remains similar for the CCD system except the SMA rods are replaced with conventional steel and the ECC can be replaced with high-strength, non-shrink grout. In this circumstance, the connection strength should match or slightly exceed the strength of the column. The chart below shows the non- Seismic P-M curve for a the same geometry as the SDD shown in Figure 8 but replacing the SMA rods with #7 rebar and replacing the ECC core with high-strength grout to match the column strength (f'c = 5 ksi). Note that the connection capacity is almost exactly matched to the column capacity. If the rebar is replaced with higher-strength rods (such as ASTM A449 or ASTM F1554-Grade 105) then the connection would be stronger than the column. Note that the connection shown in Figure 12 would exhibit the same load performance and overload characteristics as the column.

CONCLUSION

The SCS SDD represents the culmination of significant research effort to help mitigate seismic damage and allow repair of bridge plastic hinge connections. The system is designed to benefit all the stakeholders in modern bridge design:

- **The Designer:** The components can help refine a design early in the process by using components from a palette rather than multiple shapes. Design amenities and aesthetics can be added using the fairing design which can proceed nearly independently of the structural design. The appearance of the column to pier cap can even be specified or changed much later in the design process and not affect the structural design of the bridge.

- **The Contractor:** Contractors benefit from the modular nature of the system. The system will save considerable time of construction over traditional methods by allowing the pier cap/girder, column and foundation to all be poured at the same time – eliminating staging of vertical formwork and curing times. The system aids in construction alignment between the column and beams or supports. The installation minimizes the use of field grouting and connections are threaded which allows replacement of parts that have fit up problems or are damaged at any point during construction or after.

- The Owner: Faster construction times mean less disruption and lower costs. The system is designed for easy inspection both during construction and during operating life. Few of the system components (column rebar and shear pins) are hidden and those few can be readily

inspected during construction. Key attachments can be verified post-installation. The use of the fairing system can house instrumentation out of the environment and out of vandal's easy reach. The inside of the fairing can even be fitted with camera that can see the connection without anyone actually looking in an inspection port.

The Traveling Public: *Faster inspection means the bridge can be quickly verified for service if no issues are found. If the SMA needs replacement, it can be replaced and the bridge can remain in service during the replacement. Disruption to traffic is minimized..*

ACKNOWLEDGMENTS

The authors wish to thanks Dr. M "Saiid" Saiidi and Sebastian Varela at the University of Nevada Reno for their valuable experience and testing information.

NOTATIONS AND ABBREVIATIONS

SMA Shape Memory Alloy. A class of metal alloys that exhibit reversible crystal structure changes due to temperature or stress. For this application, the SMA alloy is selected to be super elastic; that is, it is able to accommodate large strains (>6%) and have little if any residual strain after the release of the load. Superelastic SMA loses energy as heat during the crystal re-arrangement leading to hysteretic energy loss. Nickel-Titanium alloy (see Nitinol below) is used here as the most common material available but Copper – Manganese-Aluminum and other alloys are being researched as lower cost alternatives.

ECC Engineered Cementitious Composite. A concrete that has very fine aggregate, relatively high cement and other pozzolan content and a large dose of polyvinyl alcohol fibers. In general, ECC has significant tensile strength and crack resistance compared to ordinary concrete.

NiTi, Nitinol A SMA consisting of nickel and titanium alloy. Nitinol is a trade name created from **Ni**ckel **Ti**tanium **N**aval **O**rdinance **L**aboratory. It was discovered during research at the Naval Ordinance Laboratory in 1959. NiTi SMA typically has yield stress of approximately 55 ksi and an E value of approximately 5500 ksi

SDD Seismic Ductility Device. A modular structural element designed to provide a plastic hinge at a beam-column connection. The device provides energy dissipation and lengthens the period of the structure. The SDD is designed to aid in construction and is easy to inspect and repair.

CCD Column Connection Device. A modular structural element designed to provide an ordinary beam-column connection. The CCD is designed to aid in construction and is easy to inspect and repair but does not utilize SMA or ECC components.

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GROUTED SPLICE SLEEVE CONNECTIONS FOR BRIDGE PIERS IN SEISMIC REGIONS: EXPERIMENTS AND ANALYSIS

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ABSTRACT

Reinforcing bar couplers are used in conjunction with prefabricated bridge elements and systems for accelerated bridge construction. Grouted splice sleeve connectors have found more applications for connections in bridge substructures because of the good construction tolerances they offer. In this research, grouted splice sleeve connectors were used in different configurations to connect half-scale precast columns to footings. Cyclic quasi-static loading was applied to two precast and one cast-in-place specimen. The connectors were located in the footing, for the first precast alternative. A debonded reinforcing bar zone was considered for the second precast alternative; the connectors were located in the column end with debonding of reinforcing bars in the footing. Debonding of reinforcing bars for the second specimen resulted in a more ductile response compared to the first precast alternative. Analytical studies were conducted to simulate the response of each specimen under lateral cyclic loading. Force-based beam column elements with fiber sections were used to construct analytical models based on plastic hinge Weighted integration. The analytical models were validated through both local and global responses with the experiments, and included low-cycle fatigue and bond-slip.

INTRODUCTION

Recent advancements in bridge construction include innovative methodologies that bring about ease of construction and acceleration of the overall project delivery time. Prefabrication of bridge elements contributes to this construction method and facilitates the whole construction process, whether the bridge is new or a replacement.

Connections between such elements in the bridge substructure are some of the most critical components in bridges constructed using accelerated bridge construction. Such connections undergo high levels of earthquake-induced deformations and stresses, while concentrated damage accumulates in a limited and localized area. Strength properties of individual grouted splice sleeve (GSS) connectors were studied in

the past (1). Tension test results provided in-depth information on the potential failure modes, ultimate load capacity, and flow of the tensile load. To date, only a few large-scale experimental studies have been conducted utilizing GSS connectors. Aida et al. (2005) and Yoshino et al. (1996) reported on the cyclic performance of large-scale specimens representing a bridge and a building subassembly, respectively (2, and 3). NCHRP report 698 evaluated several connection types applicable to Accelerated Bridge Construction (ABC), specifically for moderate-to-high seismic regions (4). The GSS connection was classified as a practical and promising connection type in that report, requiring more research on both strength and deformation properties. In particular, the inelastic behavior of such connections under cyclic loads and the sensitivity of the response to the location of the sleeves were highlighted as issues in need of further research. Haber et al. (2014) discussed the results of two half-scale specimens with GSS connectors and compared the results with a cast-in-place specimen that was cast monolithically to serve as a control specimen (5). All specimens exhibited similar performance in terms of ultimate load capacity and energy dissipation; however, the ductility capacity of the precast specimens was found to be an order of magnitude smaller than that of the control specimen. Tazarv et al. (2014) described an innovative procedure to improve the ductility capacity of the columns with the GSS connectors embedded in the column (6). The footing dowel bars were debonded within the pedestal to allow for a better spread of plasticity along the bars and consequently postpone reinforcing bar fracture.

Analytical studies are needed for a more thorough and holistic investigation of the overall aspects of the performance. To provide a basis for a better application of the experimental results in real design and construction, analytical models were developed for precast bridge piers with GSS connectors (6, and 7). These analytical models were successful in reproducing the corresponding experimental results.

As part of an extensive research program at the University of Utah, three half-scale specimens were tested under a quasi-static cyclic displacement history. Two precast column-to-footing specimens incorporated one type of GSS connector where the bars were grouted at both ends of the connector. The third specimen was built monolithically to serve as the control. The experimental program is presented in the following sections along with an analytical study that was conducted to develop analytical models for each specimen discussed herein.

EXPERIMENTAL PROGRAM

Quasi-static cyclic tests were conducted on three half-scale specimens two of which were joined by means of the GSS connectors shown in Fig. 1. These connectors were incorporated in the footing of specimen Precast-1 while the dowel bars protruded from the column end. The second precast specimen, called Precast-2, was composed of a precast column and precast footing connected by GSS connectors

which were cast at the column base; the dowel reinforcing bars protruded from the footing. These dowel bars were debonded from the concrete over an 8.0 in. region below the footing surface. The control specimen CIP was constructed monolithically without any GSS connector. Table 1 shows the test matrix and test configuration alternatives.

DESIGN AND FABRICATION OF HALF-SCALE SPECIMENS

The specimens were designed and detailed to simulate typical prototype bridges constructed in the State of Utah, following the AASHTO LRFD Bridge Design Specifications (2012), and the AASHTO Guide Specifications for LRFD Seismic Bridge Design (2011) (8, and 9). A circular configuration of column longitudinal bars and an octagonal column cross-section were adopted to facilitate the process of pre-casting the columns. Currently, the aforementioned design codes in addition to the Caltrans Seismic Design Criteria (SDC) prohibit the splicing of reinforcement, including mechanical anchorage devices, in the plastic hinge region of ductile members, for bridges located in moderate-to-high seismic areas (10). In the AASHTO Guide Specifications for LRFD Seismic Bridge

Design (2011), this would apply to Seismic Design Categories (SDCs) C and D. Thus, the preliminary design and detailing was developed for assemblies without any type of GSS connector, i.e. the cast-in-place control specimen.

The design was then adjusted to accommodate the GSS connectors within the precast specimens as needed, and essential modifications were considered accordingly.

The specimens were half-scale models of common prototype highway bridges, specifically the Riverdale Road Bridge over I-84 in Utah. The column height for all specimens was 8 ft 6 in. with a 21 in. octagonal cross-section to facilitate casting. The top 18 in. of the column was changed to a 21 in. square for testing. Six No. 8 bars in a circular arrangement and a No. 4 spiral with a pitch of 2½ in. made up the column reinforcement. The longitudinal and volumetric transverse reinforcement ratios were 1.3% and 1.9%, respectively. The footing was designed as a 6-ft long

x 3-ft wide x 2-ft deep precast concrete element and consisted of No. 8 longitudinal bars enclosed by No. 4 double hoops. The footing was designed to remain linear elastic and not undergo plastic deformations. The design inhibits shear failure from occurring in the column by using a shear span-to-depth ratio of more than 5.0 (corresponding to slender columns) along with closely spaced adequate shear reinforcement. The desirable column failure mode was set to be either flexural or splice failure. Fig. 2 shows the details and configuration of the three specimens discussed in this paper.

The GSS connectors were embedded in the footing of specimen Precast-1, with dowel bars projecting 7 in. from the precast column. A pregrout operation was conducted for this specimen using proprietary high-strength grout provided by the cast-iron sleeve manufacturer. To carry out a pregrout operation, both inlet and outlet ports of the six connectors were sealed during construction of the footing reinforcing bar cage. During erection of specimen Precast1 and before lowering and positioning the column, the grout was pumped into the wide end opening of the embedded connectors.

The GSS connectors were located in the column plastic hinge region for specimen Precast-2. A postgrout technique was implemented in which the grout was pumped into the bottom nozzle of the connectors and traveled up against gravity to fill the inside space of the connector.

TEST SETUP AND INSTRUMENTATION

The specimens were attached to the test frame using high strength bolts; the lateral cyclic load and axial load were simultaneously applied to the column top. A 120-kip servo-controlled actuator, with an overall stroke of 18 in. applied the lateral cyclic load to the precast specimens; the control specimen was tested using a 250-kip servo-controlled actuator with an overall stroke of 24 in. An axial load of 6% of the column compressive capacity was applied to simulate gravity loads. An actuator placed on top of the column, applied the compression force to a steel spreader beam which was connected to two high strength threaded rods, as shown in the test setup of Fig. 3(a).

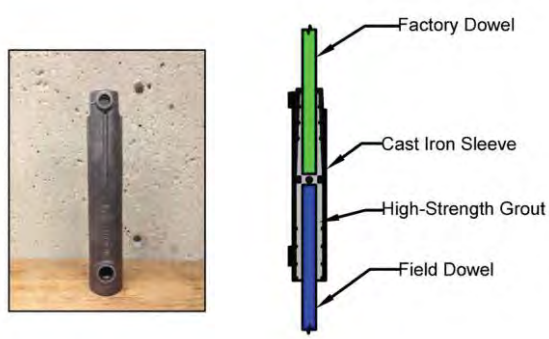

Fig. 1 Grouted splice sleeve (GSS) connector used in this study

Table 1. Test matrix

No.	Specimen	Connector Location	Other
1	Precast-1	In footing	--
2	Precast-2	In column	Debonded bar in footing
3	CIP	--	Cast-in-place

The displacement history consisted of increasing amplitudes of the predicted column yield displacement (11). Two cycles were employed for each displacement step to the east and west. Fig. 3(b) shows the drift history versus number of cycles. The drift ratio is defined as the lateral displacement of the column top, where the actuator applied the lateral load, divided by the distance from top of the footing to center of the lateral load application.

Linear variable differential transformers (LVDT) were used to study the curvature distribution along the column end.

These LVDTs were mounted to the column end, over an approximately 30-in. region, to measure the relative vertical displacements between the sections and provide data for curvature analysis.

SUMMARY OF TEST RESULTS

Test-day Material Properties

Tension tests of reinforcing bars were conducted along with compression tests on concrete cylinders and grout cubes for each specimen. No. 8 ASTM A706 Grade 60 reinforcement was used as longitudinal column bars which had an average yield strength and ultimate strength of 68 ksi and 93 ksi, respectively. Table 2 contains the compression test results for the concrete and grout for both the 28th day of curing and test-day of the specific test.

Hysteretic Performance and Observations

The overall response of the three specimens was good as implied by the wide and stable hysteresis loops that indicate a relatively high energy dissipation capacity, as shown in Fig. 4. The damage states mark three major events during the cyclic tests: end of flexural crack formation and initiation of cover spalling, yield penetration, and fracture of column longitudinal reinforcing bars.

Specimen Precast-1 had a stable performance up to the first cycle of the 7% drift ratio, during which the column east reinforcing bar fractured at a section 2 in. above the column-to-footing interface, due to low-cycle fatigue. There was no sign of excessive in-cycle or cyclic strength deterioration before the fracture of the east reinforcing bar. The test was terminated after completion of the 7% drift ratio as a result of a

35% strength reduction. The displacement ductility of this specimen was found to be 6.1; this quantity was calculated using the average cyclic envelop of the force-displacement response and widely accepted procedures based on the concept of equal energy of an idealized elasto-plastic system (12).

The overall hysteretic response of Precast-2 indicated an entirely satisfactory and ductile performance. The

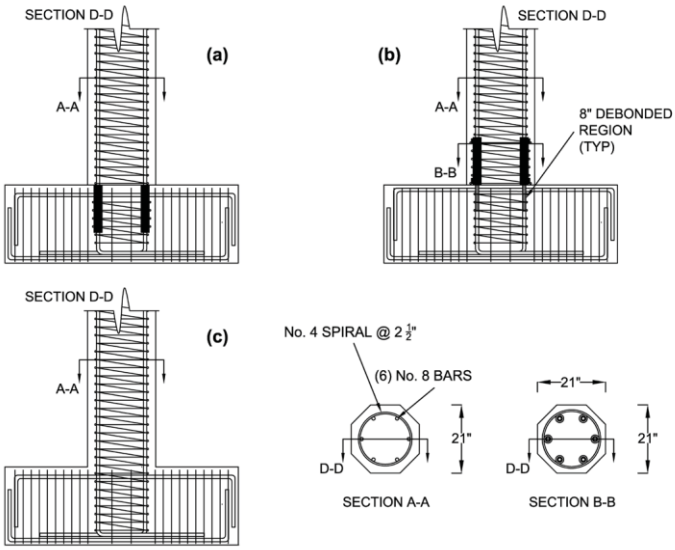

Fig. 2 Details of specimens: (a) Precast-1, (b) Precast-2, and (c) CIP

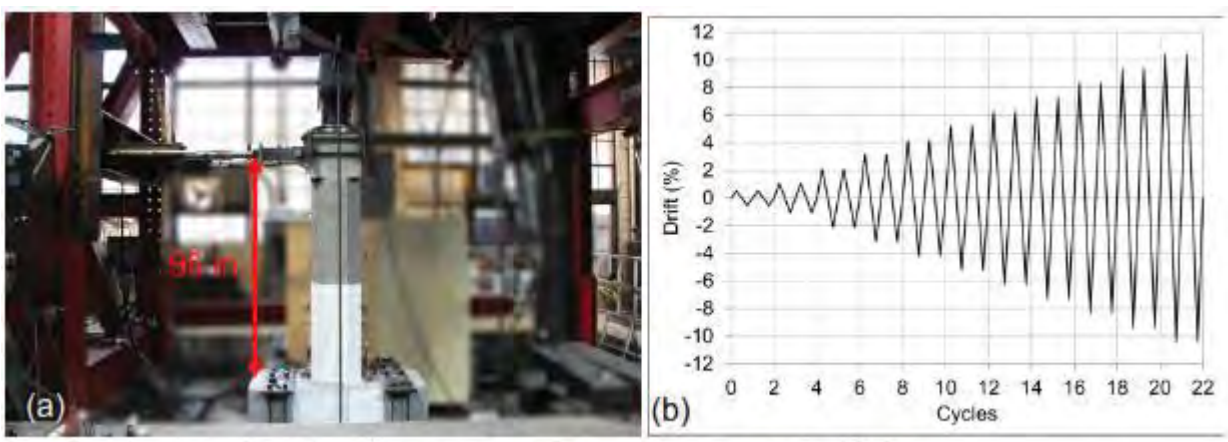

Fig. 3 Experimental setup: (a) Test configuration, and (b) Drift history

hysteresis loops of this specimen were wide and stable with minimal strength deterioration up to the first cycle during the 8% drift ratio, when the extreme east column reinforcing bar fractured ½ in. below the column-to-footing interface.

The deboning of footing dowel bars inside the footing of this precast specimen resulted in an extended performance life, compared to Precast-1. The displacement ductility of this specimen was found to be 6.8 implying that a more ductile response was achieved because of the debonded reinforcing bar region inside the footing.

The control specimen CIP had stable hysteresis loops with a slight level of strength degradation that was caused by spalling of the relatively large unconfined concrete cover. To keep the sectional configuration of column reinforcing bars identical among all specimens, specimen CIP had the thickest cover so that the location of the column longitudinal bars remained unchanged. As expected, specimen CIP had the longest performance life compared to the two precast specimens. For this specimen, the extreme west column reinforcing bar fractured towards the end of the second cycle of the 8% drift ratio, at a section 1 ½ in. above the column-to-footing interface. Subsequently, the extreme east column reinforcing bar fractured during the first cycle of the 9% drift ratio, 2 in. above the interface achieving a displacement ductility of 8.9. More detailed discussion on the experimental results could be found in the work of Pantelides et al. (13).

ANALYTICAL STUDY

The application of prefabricated bridge elements and systems (PBES) as part of ABC is becoming a method of choice in many States in the US. In regions of high seismicity, the connections between such prefabricated elements are critical for the structural integrity of the bridge, especially if the GSS connectors are used at moment connections.

The results of the large-scale experiments discussed in the preceding sections, as well as other experimental studies, suggest that there are significant performance differences between the precast and conventional cast-in-place construction, under simulated seismic loads (5, 6, and 14). Therefore, reliable analytical models capable of predicting the response of precast columns with GSS connectors would be a valuable analysis and design tool for researchers and engineers.

Analysis Objectives

The objectives of the analytical study are as follows: (a) to develop an analytical model capable of replicating the experimental results, accounting for inherent differences on the precast specimens; and (b) to apply the model to a general evaluation/design condition with actual design details.

Nonlinear analysis of reinforced concrete components is complex. To develop an efficient analytical approach, it is helpful if simplified yet accurate analytical models are incorporated. The proposed

analytical model is composed of a beam-column (stick-based) element with distributed plasticity. In the distributed plasticity models or fiber models, nonlinear material behavior can occur at any element cross-section. These-cross sections are discretized into a finite number of fibers and pertinent uniaxial material stress-strain relationships are assigned to each type of fiber.

Table 2. Compressive strength of concrete and grout (ksi)

Specimen	Concrete		Grout	
	28-day	Test day	28-day	Test day
Precast-1	3.9	5.5	11.1	13.5
Precast-2	6.7	8.4	15.6	14.6
CIP	5.2	6.7	NA	NA

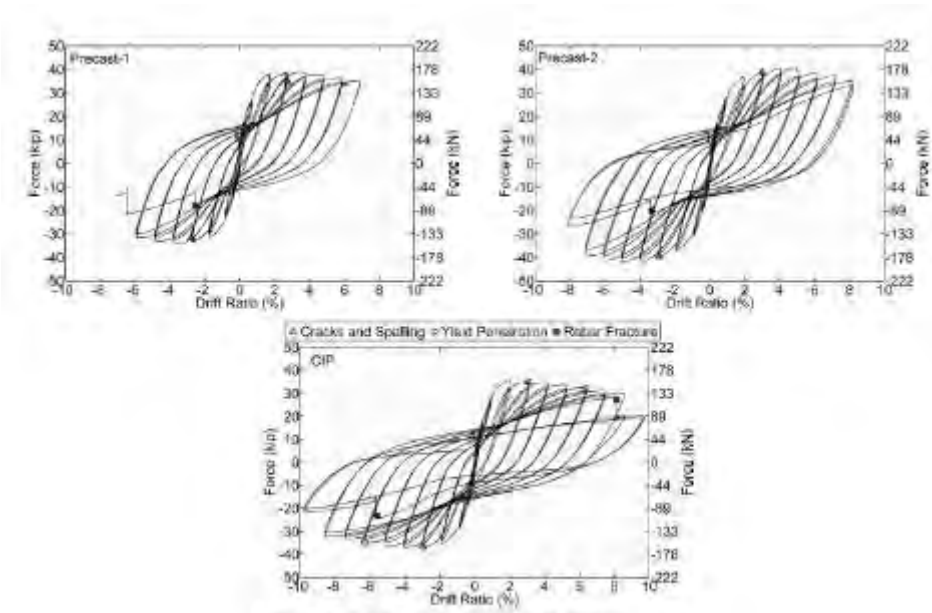

Fig. 4 Hysteretic response of specimens

Numerical integration is evaluated along the element to obtain the global response.

Description of Proposed Analytical Model

The proposed analytical model was developed using the force-based beam-column element object in the OpenSees framework (15). This includes incorporating a certain number of integration points (typically between four and six) along the element; however, when dealing with a softening system, the response of the element will not converge with an increase in the number of integration points (16). This is because the deformations are localized at one integration point only (located at the critical section along the beam-column element) leading to a loss of objectivity, therefore, the response varies as the integration weight of that single integration point changes when changing the total number of integration points used in the element. To mitigate this issue, the plastic hinge integration scheme introduced by Scott and Fenves

(2006) was adopted in this study so that the proposed model could be used for a wide range of reinforced or precast concrete components (16). A modified version of this element that can be found in the element library of OpenSees was used in this study. This modified element allows for spread of nonlinearity beyond the plastic hinge region unlike the original formulation that confines the nonlinear behavior to the plastic hinge region, while the interior segment of the element invariably remains linear elastic. One major advantage of using the force-based beam-column element with the plastic hinge integration scheme is that the user can specify a predefined plastic hinge length using empirical equations available in the literature or based on direct observations from experiments.

The proposed analytical model was developed using an iterative procedure to obtain the unique plastic hinge length that would result in an acceptable response under static cyclic loading. Two criteria were incorporated to achieve a high level of accuracy:

(1) Global response, which refers to the overall response of the model under static cyclic loading. In particular, the hysteretic performance of the proposed model is compared with that of the corresponding specimen to verify if strength and stiffness are in good agreement. A quantitative measure for this acceptance criterion is to compute the hysteretic energy per drift ratio and make a thorough comparison.
(2) Local response, or sectional response includes validation of the sectional performance of the proposed model with reference to that of the corresponding specimen. In this study, a moment-curvature approach was selected to study and compare the local response. A secondary local response criterion is fracture of reinforcing bars due to low-cycle fatigue which was the cause of failure for all specimens in this study, based on experimental observations. This is also included in the proposed model as an indication of ultimate displacement capacity.

To initiate the process, a trial value of plastic hinge length (L_p) is selected and the analytical model is examined to meet both criteria. If any of the criteria is not met, a new value of L_p is specified until satisfactory results are achieved.

This procedure is based upon transformation of the precast specimen into an idealized equivalent cast-in-place model with a plastic hinge region that is capable of reproducing similar global and local performance.

The application of GSS connectors for connecting precast columns to precast footings altered the load transfer mechanism in the subassemblies discussed in this paper. This load transfer is highly based upon the bond between the reinforcing bars and surrounding high-strength grout. This would signify the

importance of bond-slip in the response of such subassemblies. The bond-slip characteristics of all specimens were included in the proposed analytical model.

Bond-slip

Bond-slip of reinforcing bars is an important phenomenon that may influence the response of reinforced/precast concrete components during extreme seismic events. This is particularly significant at the interface of bridge piers and footings, cap beams, or pile caps due to a large inelastic demand near the joint region. Past research studies showed that bond-slip affects both the local and global response of reinforced concrete members (17). Hence, it needs to be considered in the analysis of reinforced/precast concrete components.

A nonlinear one-dimensional finite element model was developed to investigate the bond-slip of reinforcing bars grouted inside the GSS connectors. This model was developed in OpeSees, based on the general schematic model described in the work of Raynor et al. and Steuck et al. (18, 19). The analytical model was composed of a series of discretized reinforcing bars (using nonlinear truss elements) connected to bond-slip springs (represented by zerolength elements) at each node. As shown in Fig. 5(a), which presents the schematic of the bond-slip model, both confined and unconfined regions were included in the model. The unconfined region represents the grout cone that forms near the two ends of the GSS connector because of localized stresses around the unsupported ends; this would result in an inferior bond between the reinforcing bar and the surrounding grout. Both unconfined and confined bond constitutive laws were taken from the experimental results of bars grouted in corrugated steel ducts, as this was the closest condition to GSS connectors (19). Such experimental data is not yet available for GSS connectors. Fig. 5(b) shows the result of a tensile test on the GSS connector specimen GS3 from Haber et al. (2013), along with the result of the proposed model developed in this study (20). A discretization study revealed that 50 bond-slip springs for each dowel bar would be sufficient to obtain adequate performance; this figure shows that the overall response of the proposed model compared the experiment is excellent.

To implement the results of the proposed one-dimensional bond-slip model in the analytical model of the half-scale specimens, a pseudo stress-strain relationship is derived for the reinforcing bar in the plastic hinge region, using the output results of the one-dimensional bond-slip analytical model. The pseudo stress is defined as the output force in the outermost bar element divided by the area of reinforcing bar, and the pseudo strain is defined as:

$$\varepsilon = u/L_{\mathrm{p}} \tag{1}$$

where, u is the total end displacement of the connector model, and L_p is the plastic hinge length of the half-scale analytical model. For the CIP, however, uu refers to the end displacement of the reinforcing bar embedded in conventional concrete. The bond-slip constitutive law for the case of CIP was taken from Eligehausen et al. (21).

Low-cycle Fatigue

Based on the experimental results, two extreme column bars of CIP fractured due to low-cycle fatigue as a result of successive bending and re-straightening during each drift ratio. A premature reinforcing bar fracture occurred for Precast-1, while debonding of bars in the footing of Precast-2 resulted in a delayed premature fracture. To account for low-cycle fatigue in the proposed model, ReinforcingSteel material in OpenSees was assigned to reinforcing bars in the columns. This material is capable of predicting the low-cycle fatigue life of reinforcing bars, by using the Coffin-

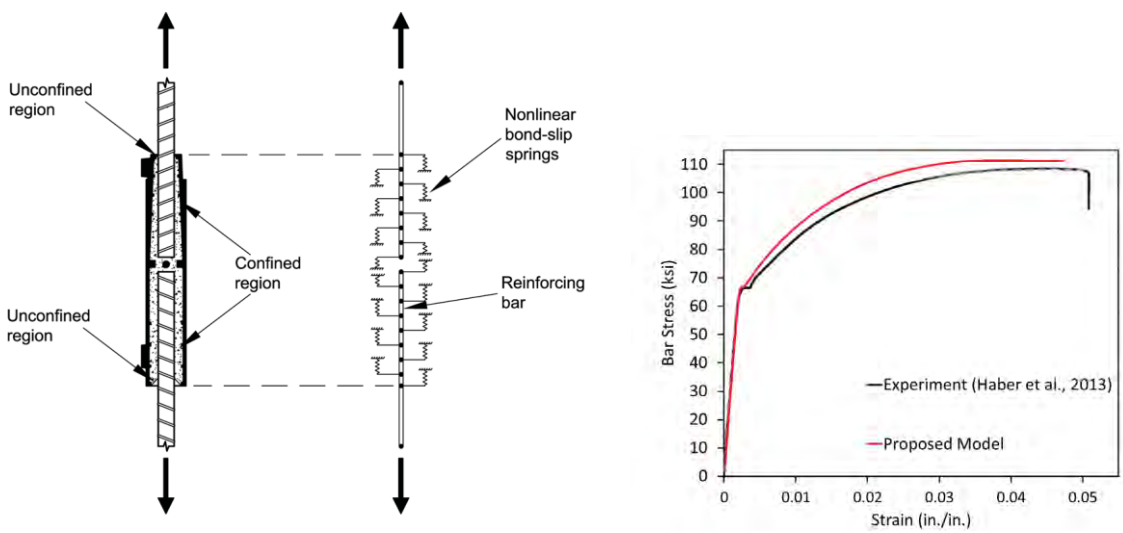

Fig. 5 (a) Schematic of analytical bond-slip model, and (b) Validation of proposed model

Manson expression along with a cumulative linear damage rule, as shown in Eq. (2) and Eq. (3), respectively (22, 23, and 24).

$$\varepsilon^p = C_f(2N_f)^{-\alpha} \tag{2}$$

$$D_f = \frac{1}{\sum_{i=1}^{n}(2N_f)_i} \tag{3}$$

where, ε^p is plastic strain amplitude, C_f and α are material constants, $2N_f$ is the number of half-cycles to failure. In this study, C_f and α values of 0.26 and 0.506 were used, respectively, based on the experimental investigation of the ReinforcingSteel material developers (25).

Model Layout

The proposed model is composed of two nodes and one force-based beam-column element with the plastic hinge integration scheme. A unique plastic hinge length was obtained for each specimen following the iterative process described previously. For the case of CIP, a plastic hinge length of 12.0 in. was found to be adequate. This value correlates well with the empirical expression found in the work of Panagiotakos and Fardis (2001) for cyclic loading, when reinforcing bar pullout is not present (26). The proposed model estimated plastic hinge lengths equal to 8.0 in. and 10.0 in. for Precast-1 and Precast-2, respectively. The schematic layout of the proposed model is shown in Fig. 6 for the three specimens. The lateral cyclic displacement and axial monotonic load are applied to the top node, as shown.

The octagonal column was approximated by a circular section of equal cross-sectional area, to simplify the sectional discretization process. There were 40 circumferential and 20 radial subdivisions for the confined core concrete, and

40 circumferential and 5 radial subdivisions for the unconfined cover concrete. Concrete04 material was used for both confined and unconfined concrete with appropriate stress-strain properties. ReinforcingSteel was used to define the uniaxial pseudo stress-strain relationship of the reinforcing bars in the plastic hinge region, and the original uniaxial stress-strain relationship outside the plastic hinge region. The validation process is presented in the flowchart of Fig.7.

ANALYSIS RESULTS AND COMPARISON

Global Response

The hysteresis response and hysteretic energy dissipation capacity of the three specimens, up to the last completed drift ratio, including both the experimental results and the results from the proposed analytical models is shown in Fig.

8. The proposed model correctly identified the cycle and drift ratio within which the bar fracture occurred. The results from the proposed model are in close agreement with the experiments. The strength and stiffness parameters are satisfactory, while the hysteretic energy of the proposed model is within an acceptable range. There is only a 10.6%,

7.8%, 4.6% difference between the experimental energy dissipation capacity and that of the analytical model at 5% drift ratio, for CIP, Precast-1, and Precast-2, respectively.

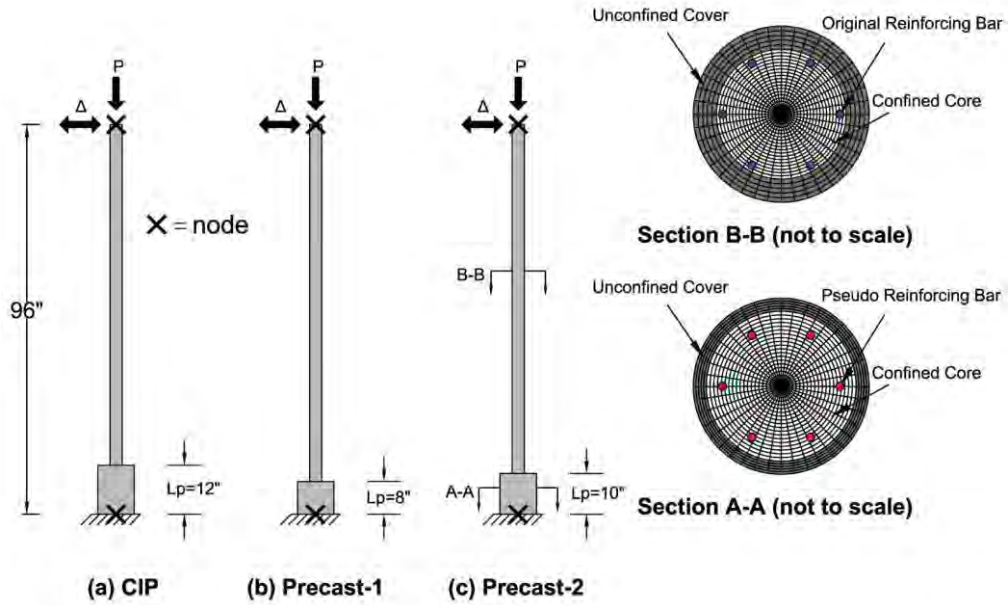

Fig. 6 Schematic layout of proposed model

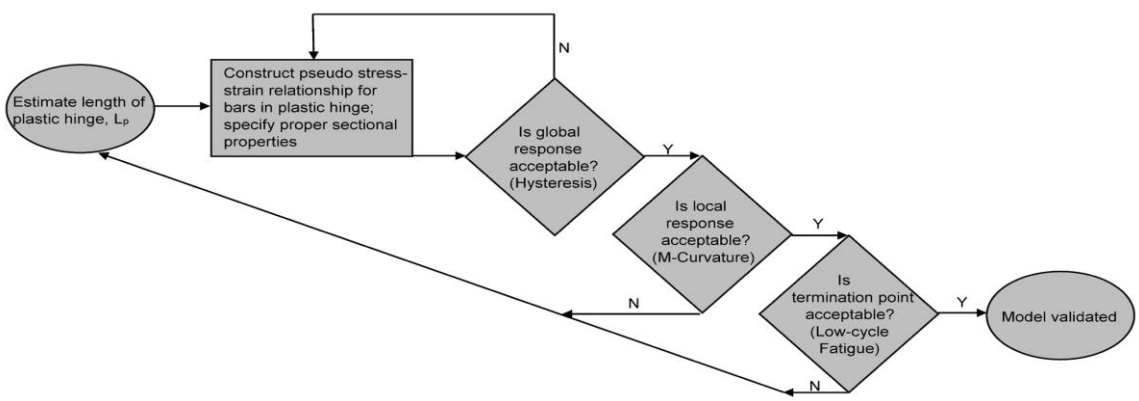

Fig. 7 Validation process

Local Response

The moment-curvature envelope for the push direction of each specimen is presented in Fig. 9, up to the end of the 6% drift ratio, when LVDTs were removed from the specimens. The curvature values are normalized by multiplying the average curvature by the column dimension of 21 in. The corresponding analytical results suggest that the proposed model is capable of replicating the sectional response as well as the global response. At 6% drift ratio, there is an 11.1%, 4.1%, and 10.8% difference between the peak curvature capacity obtained from the experiments and that of the analyses for CIP, Precast-1, and Precast-2, respectively.

CONCLUSIONS

The experimental evaluation of column-to-footing tests conducted in this research provided qualitative and quantitative measures to evaluate precast and cast-in-place specimens under quasi-static cyclic loads. The control specimen, CIP, had a very good hysteretic response with ductile performance. Extreme column bars fractured during the 8% and 9% drift ratio for CIP, because of low-cycle fatigue. Both precast specimens had ductile performance comparable to the performance of CIP, up to the failure point that was caused by premature reinforcing bar fracture.

Testing of Precast-1, with grouted splice sleeve connectors inside the footing, was terminated at the end of the 7% drift ratio. Precast-2, with grouted splice sleeve connectors in the plastic hinge region of the column and debonded bars in the footing, had a longer performance life, as the extreme dowel bar fractured during the 8% drift ratio.

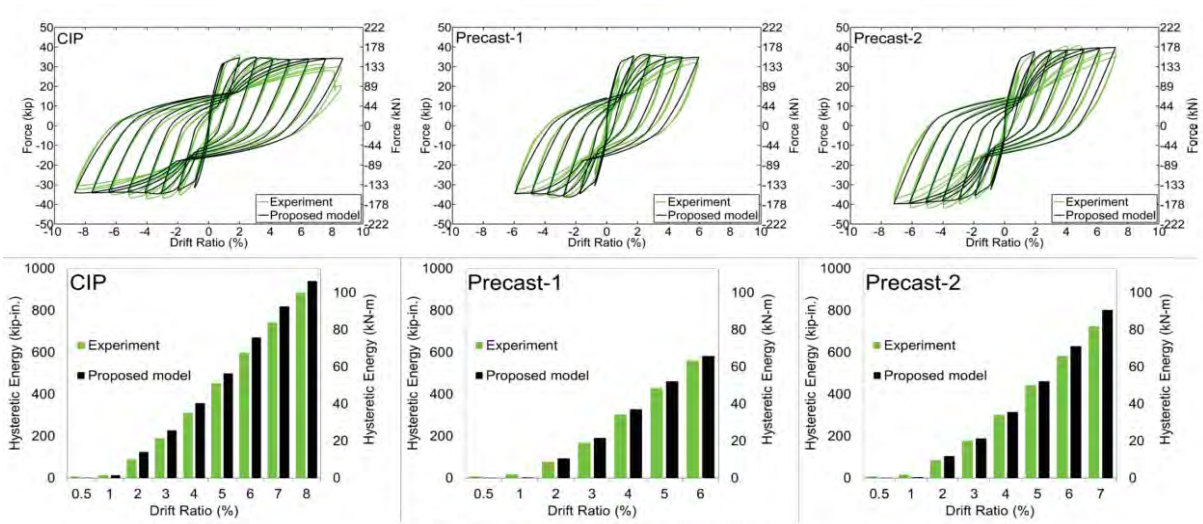

Fig. 8 Comparison of global response

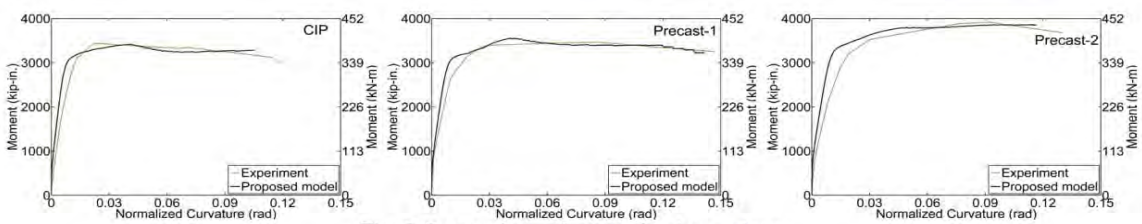

Fig. 9 Comparison of normalized curvature

A two-dimensional analytical model was developed based on transformation of precast column with grouted splice sleeve connectors, to an idealized equivalent cast-in-place column with a plastic hinge region that is capable of simulating both the global and local (sectional) response. A force-based beam-column element type with plastic hinge integration scheme was used in an iterative procedure to

determine the unique plastic hinge length of the equivalent systems, and reproduce the experimental results. The proposed model that included the bond-slip of reinforcing bars in addition to the low-cycle fatigue was successful in replicating both the global and local response. To account for the bond-slip effects, a pseudo stress-strain relationship was obtained from the results of a one-dimensional analytical model. The computed uniaxial pseudo stress-strain was then incorporated into the model over the plastic hinge region of each specimen. The validated analytical model had a plastic hinge length of 12.0 in. for CIP, while a plastic hinge length equal to 8.0 in. and 10.0 in. was obtained for the equivalent model of Precast-1 and Precast-2, respectively.

ACKNOWLEDGMENTS

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NOTATION

$2N_f$=number of half-cycles to failure

C_f=low-cycle fatigue material constant

D_f=fatigue damage

L_p=length of plastic hinge

u =end displacement of one-dimensional connector model

α=low-cycle fatigue material constant

ε=pseudo strain

ε_p=plastic strain amplitude

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SEISMIC ACCELERATED BRIDGE CONSTRUCTION WITH LOW-DAMAGE SEGMENTAL BRIDGECOLUMNS INCORPORATING DAMAGE-RESISTANT JOINTS

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INTRODUCTION

Bridge substructures designed and constructed adopting conventional seismic design methodologies provide ductility and energy dissipation capabilities through formation of plastic hinges at the column ends. Although this design philosophy ensures life safety, following a design earthquake, bridges will sustain large damage, which results in extensive repair costs and downtime. Moreover, conventional cast-in-place techniques require large construction times. In the last two decades, several efforts have been made to develop systems that combine construction rapidity with improved seismic performance. These efforts have led to several substructure systems incorporating segmental or monolithic rocking columns with unbonded post-tensioning (PT) [1-5].

The present study introduces a new bridge substructure system incorporating bridge segmental columns with flexible damage-resistant (DR) joints/segments and internal unbonded post-tensioning. The DR joints are (flexurally) deformable enough to accommodate large lateral deformations without damage, and (axially) stiff enough to limit the axial deformations due to initial post-tensioning and gravity loads. A computational parametric study is conducted for a cantilever column with a DR segment at the bottom via three dimensional (3D) finite element models created in the general-purpose nonlinear finite element analysis (FEA) program ABAQUS [6]. The column is subjected to monotonic and cyclic lateral loading. Two polyurethane (PU) elastomers and various geometries (height and diameter) of the DR segment are explored. Compressive uniaxial material testing is conducted for the calibration of material models of the PU elastomers.

COMPRESSION TESTS AND MODEL CALIBRATION

The elastomers used in this study are available from BASF [7] as Elastocore and Elastoc ast. Cylindrical samples of length of 4 in.

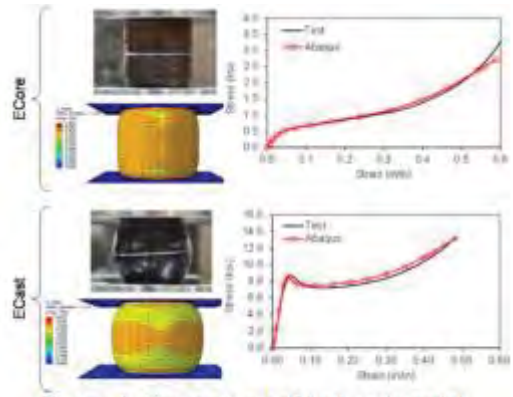

Figure 1. Comparison of test and model results.

and diameter of 2 in. were tested in the Structures andMaterials Testing Laboratory (SMTL) of the University of Colorado - Boulder under uniaxial compression loading at a crush rate of 0.05 in/min, in accordance with the ASTM D695 [8]. Finite element models were developed in ABAQUS [6] and Marlow hyperelastic material model is calibrated using compression test results. A comparison of computed and measured stress vs. strain curves is shown in Figure 1. The same figure also presents a comparison of the deformed shapes of the computational models and test samples for a strain of 40%.

PARAMETRIC STUDY OF SEGMENTAL COLUMN WITH DR JOINT

The reference model for this study was a cantilever segmental concrete-only column (specimen SC-2) tested by Motaref et al. [7] under quasi-static lateral cyclic loading. Variations of this model were obtained by replacing its lower concrete segment (height of 20 in. and diameter of 16 in.) with a DR segment made of PU. A FE model of the reference column was prepared and verified using available experimental data [7], as shown in Figure 2. This model was then modified to incorporate DR segments at the bottom. Monotonic and cyclic pushover analyses were conducted for columns with ECore and ECast PU segments for a constant vertical load of 80 kips. Under lateral loading, the main deformation mechanism of the ECore column was flexure without rocking, whereas, the ECast column exhibited flexure and rocking at the bottom (see Figure 3).

The ECast column developed a low stiffness but similar strength with the reference concrete column (~20 kips at a 10% drift ratio). On the contrary, the ECore column developed a very low lateral stiffness and strength (~3 kips at a 10% drift ratio) and remained elastic (see Figure 3). The cyclic lateral force vs. drift ratio response of the ECast and ECore columns are compared to the RC column in Figure 4 (a). The response is nearly

symmetric. The cumulative energy dissipation and the relative self-centering efficiency (RSE), which represents the portion of the peak deformations that are recoverable, are shown in Figure 4 (b) and (c).The distribution of the compressive damage index (DAMAGEC) on the deformed shape of specimens with various hi/Di and Dj/Dc ratios (hj, Dj and Dc: DR segment height, DRsegment diameter, and column diameter, respectively) ratio at the maximum drift ratio the analysis could converge is shown in Figure 5 and Figure 6, respectively. According to Figure 5, as the height of the ECore segment decreases, the deformation mechanism changes from pure flexure to a combination of flexural deformation and rocking at the bottom joint accompanied by increased damage in the concrete at the interface above the DR segment. In the ECast column, as hj/Dj decreases (with Dj/Dc = 1), the deformation mechanism changes from rocking at the bottom joint to rocking at the interface above the DR segment followed by further concrete damage (Figure 5). For the ECore column, reducing hj/Dj (with Dj/Dc

=1), increases the lateral stiffness and strength of the columns (Figure 6a). For the ECast columns though, increasing hj/Dj (with Dj/Dc = 1) has negligible effects on stiffness and strength, unless hj/Dj = 0.5 for which strength deterioration is observed

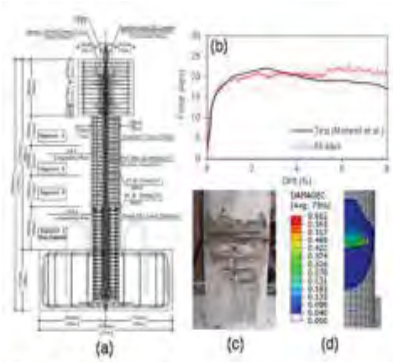

Figure 2. (a) Reference segmental columns [9]; (b) Comparison of lateral force versus drift ratio (c) Damage at 10% drift (Photos from Motaref et al. [9]), (d) Computed damage at 10% drift

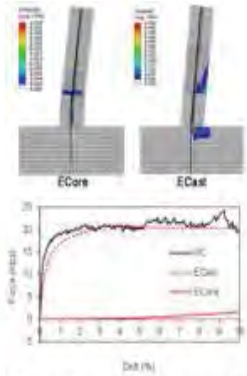

Figure 3. Deformed shape under lateral loading after application of vertical loads for Ecore and Ecast

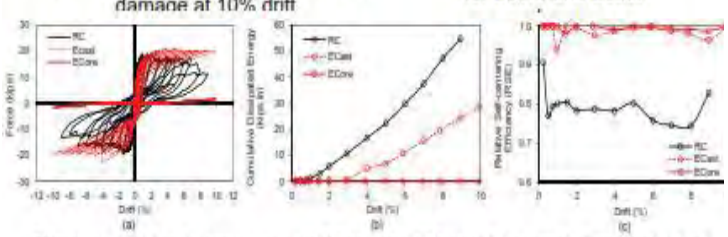

Figure 4. (a) Cyclic response of ECast and ECore; (b) cumulative dissipated energy; (c) RSE versus drift ratio

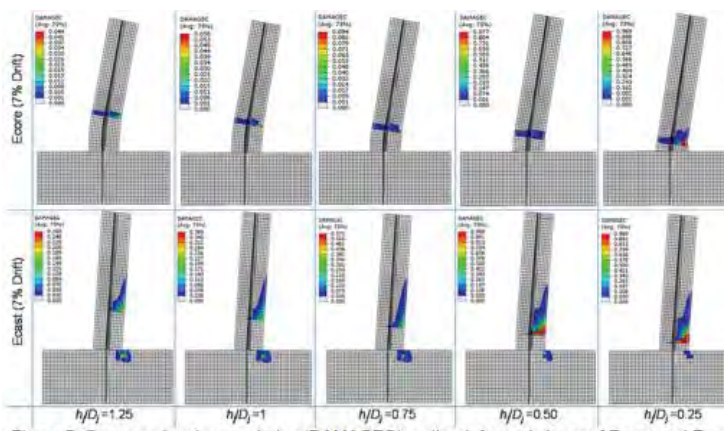

Figure 5. Compressive damage index (DAMAGEC) on the deformed shape of Ecore and Ecas columns

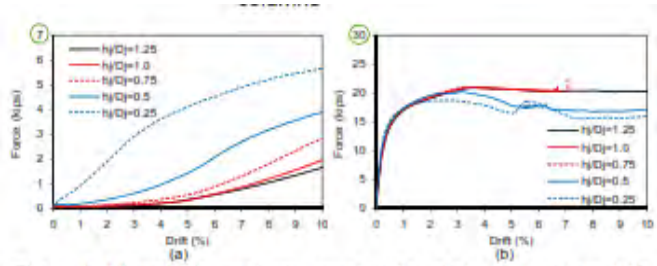

Figure 6. Monotonic analyses results of models with various h_j/D_j ratio and D_j/D_c = 1: (a) ECore; (b) ECast

beyond a drift ratio of 2-3% due to the change in the deformation mechanism of the column (Figure 6 (b)).

By increasing the diameter of the ECore segment, the deformation mechanism changes from pure flexure to a combination of flexure of the ECore segment and rocking at the joint above the ECore segment, while the damage concentration increases at that joint.

In the ECast column, as Dj/Dc reduces, the deformation mechanism changes fromrocking at the bottom joint to rocking at the joint above the ECast segment, which is followed by increased concrete damage. For both the ECore and ECast columns, the initial stiffness and peak lateral strength increases with Dj/Dc, as shown in Figure 7.

SUMMARY AND CONCLUSION

A new segmental bridge column system employing DR flexible joints made from PU elastomers to control damage at large deformation, while maintaining reasonable stiffness, is proposed. A parametric study was conducted via a three dimensional finite element (FE) model to assess the response properties of the proposed bridge column for two PU elastomers and various geometric properties of the flexible joint. Compressive uniaxial material testing was conducted for the PU elastomers to calibrate FE models. Major findings include:

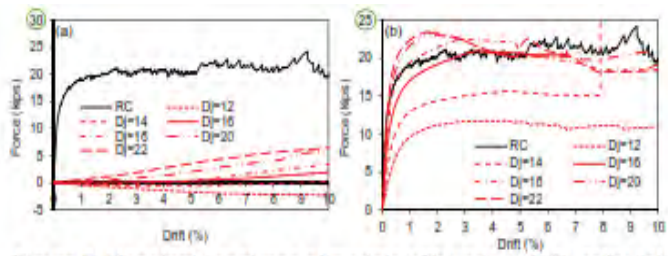

Figure 7. Monotonic analyses of models with various D_j/D_c ratio: (a) ECore; (b) ECast.

- Columns with DR segments made of hyper-elastic materials, such as PUs, can accommodate large deformation (drift ratios in the order of 10%) without damage.

- By designing the post-tensioning system to remain elastic, complete self-centering can be achieved.

- The geometric properties of the DR segment are critical for the column response. Specifically, damage in the concrete above the DR segment can be reduced (even for stiff PUs) when the diameter of the DR segment is smaller than the diameter of the column. Also, columns with very short DR segments experience more concrete damage whereas, in the case of very flexible DR joints, e.g. ECore, the column lateral strength increases with shorter segments.

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INNOVATIVE ABC COLUMNS WITH COPPER-BASED SMA AND ECC FOR SEISMICALLY ACTIVE REGIONS

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INTRODUCTION

Failure of standard reinforced concrete (RC) highway bridges during an earthquake can have a prominent economic and social impact. Monetary losses due to bridge repair, demolition, or replacement and business disruption can greatly affect the economy of a region while bridge downtime hinders post-disaster response efforts. Although accelerated bridge construction (ABC) techniques provide easier, safer, and faster construction practices, transportation agencies are still hesitant about using ABC in seismically active regions. This is because there are concerns about how well ABC columns and their connections will perform under seismic loading. It has been thought that ABC columns in high seismic zones should emulate Cast-In-Place (CIP) RC columns, which are designed to experience plastic behavior and dissipate energy during an earthquake. However, this behavior comes at the expense of damage, permanent drifts and loss of functionality since designing RC columns to stay in service after a strong earthquake is rare and cost prohibitive.

It would be desirable to find a way to design ABC columns for high seismic zones that were able to resist intense earthquake loading and yet remain functional with minimal or no damage and low permanent drifts. A way to achieve this is to use smart materials that are able to accommodate large earthquake displacements without undergoing the typical severe damage seen in RC members. Superelastic Shape

Memory Alloy (SMA) bars can be used in lieu of the mild steel reinforcing bars in the plastic hinge of a column. The superelastic property of SMA allows the bars to experience very large strains (in excess of

6%) with minimal plastic deformation, which results in a very effective self-centering mechanism for the column even under large drift demands. Similarly, Engineered Cementitious Composite (ECC) can be used to replace the concrete in the plastic hinge to mitigate damage and loss of capacity. This is thanks to the fibers and special admixtures in ECC that give it superior ductility in tension and compression. The effectiveness of SMA and ECC to resist intense earthquake loading while maintaining bridge column

functionality was demonstrated by Saiidi and Wang (1), Saiidi et al. (2), Cruz-Noguez and Saiidi (3), among others. Most previous applications of SMA for bridge columns have utilized Nickel-Titanium (NiTi) SMA, which is expensive due to the high cost of Titanium and because it is hard to machine. An emerging Copper-Aluminum-Manganese (CAM) SMA has been also shown to provide good seismic performance at only a fraction of the cost of NiTi (Varela and Saiidi (4), Araki et al. (5), Shrestha et al.(6)).

This extended abstract outlines the main observations from experimental studies conducted at the University of Nevada Reno on ABC columns using CAM SMA and ECC. The overall objective of this research was to develop novel ABC columns that are able to resist extreme earthquake loading without the post-earthquake functionality shortcomings of CIP construction with conventional steel and concrete materials. Another objective was to adopt the concept of Design for Disassembly (DfD) so bridge columns could be detached to facilitate component reuse and material recycling at the structure's end-of life. Since DfD reduces energy consumption and CO_2 footprint during material extraction and manufacture (Webster(7)), it can lead to a sustainable bridge engineering practice. This is a key for RC bridges since CO_2 emissions from cement manufacture account for about 5% of all global anthropogenic carbon dioxide sources, Worrel et al. (8).

CU-CIP COLUMN MODEL

Varela and Saiidi (4) tested for the first time a large-scale CIP RC bridge column model (Cu-CIP) incorporating CAM SMA and ECC in the plastic hinge under simulated earthquake motions. The ¼-scale model was subjected to near-fault motions from the 1994 Northridge California, Earthquake which has beenfound to cause large residual drifts in RC columns due to its impulsive characteristics, Phan et al.(9).

The apparent damage state of the model after the last earthquake motion, equivalent to 450% times the'design' level earthquake (peak ground acceleration=1.31 g), consisted of multiple fine tensile cracks and minor compressive flaking of the ECC in the plastic hinge that could be easily repaired. The loss of capacity in the column was minimal despite that two out of 10 CAM bars broke during the last motion. Strain and curvature data showed that all nonlinear behavior took place in the plastic hinge zone only. CAM SMA and ECC significantly reduced the extent of apparent damage in the column, similarly to previous findings on models with ECC and NiTi SMA. The high self-centering provided by CAM SMA was evident since the residual drift following testing was less than 0.5%, which is very low considering that the model experienced a maximum drift close to 12%. It was concluded that using CAM SMA and ECC in the plastic hinge region of RC columns in high seismic zones could be a feasible alternative to keep bridges functional after strong earthquakes.

DFD COLUMN MODELS

Although the seismic performance of the Cu-CIP model was remarkable, the DfD concept described in detail in Varela and Saiidi (10) is more in line with ABC principles. A ¼-scale DfD column model utilizing a precast plastic hinge made of ECC with unbonded CAM SMA bars was tested dynamically under the same motions used for Cu-CIP. Since it was desired to study the effect of reusing column components, the column model was tested, disassembled, and then reassembled and retested. The model was comprised of four main components: 1) an upper concrete-filled CFRP tube column body with protruding longitudinal bars at the bottom; 2) a precast plastic hinge made of ECC; 3) dog-bone shaped CAM SMA bars with threaded ends; 4) and a precast footing incorporating threaded dowels. The column body and footing were completely detachable from the plastic hinge element and were designed to remain elastic while the base element and inner SMA bars dissipated energy with minimal or no damage. The 'virgin' model was subjected to a maximum simulated earthquake motion with a peak ground acceleration (PGA) of 0.76g that caused a maximum drift of 6.7%, while the reassembled model was subjected to a motion with PGA of 0.83g resulting in a maximum drift of 7%. The measured residual drifts on the 'virgin' and reassembled models were 0.3% and 0.13%, respectively, which could be safely accommodated by a bridge system. The loss of lateral and vertical load-carrying capacity for both models was minimal. There was no damage in any component other than the plastic hinge, and damage in this element was limited to repairable spalling and cracking of unconfined ECC as shown in Fig. 1. None of the SMA bars were broken. The reassembled model was able to reach the same capacity of the 'virgin' model but exhibited lower stiffness due to cumulative damage.

Figure 1. Plastic hinge damage at the end of the last earthquake motion. Top: 'virgin' model; Bottom: reassembled model.

CONCLUSIONS

Based on the experimental and analytical studies summarized in this article, the following conclusions were drawn:

1. CAM SMA and ECC combination is effective in reducing damage, loss of capacity, and residual drift in bridge columns subjected to strong earthquakes.

2. The DfD concept developed in this study is compatible with ABC and could be used to keep bridges open to traffic even after strong earthquakes. This would reduce monetary losses associated with bridge downtime, repair, and future replacement.

3. DfD facilitates reuse and recycling of column components, thereby decreasing energy consumption and carbon footprint from the material extraction and manufacturing processes and thus fostering sustainability of bridges.

ACKNOWLEDGMENTS

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A CLASS OF V-CONNECTORS FOR BRIDGE'S DECK-PIER AND PIER-FOOTING JOINTS WITH COMBINED ADVANTAGES OF INTEGRATED DESIGN AND SEISMIC ISOLATION WHILE ENABLING ABC

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CONCEPT AND PRODUCT

This article discloses a class of innovative devices, termed "V-Connectors", Fig. 1, which can be used as the joints between pier and superstructure or between pier and footing in a bridge, with the unique advantages in accelerated bridge construction and the robustness for required seismic-resistance.

Fig. 1 illustrates the basic configuration of such a V-connector, which is an assembly of the five basic elements to be manufactured: two V-shape guiding tubes (VGT) that are respectively mounted into the two connected structural parts, a bridge's deck and pier in this figure; a vertical stabilization-pin, abbreviation "SBP", with its two ends inserted respectively into the two VGTs; a damping cone around the pin within the lower VGT, and a washer. While as a seal to prevent dirt fell into VGT, the major function of the washer is to provide selectable friction-induced energy dissipation when a bridge is struck by earthquakes, see the hysteric loops on the right of Fig. 1. However, it is optional because the friction between the contact-surface pair of the connected structural parts may perform the same function. Also, the damping core is also optional if the friction-induced dissipation is sufficient for the system.

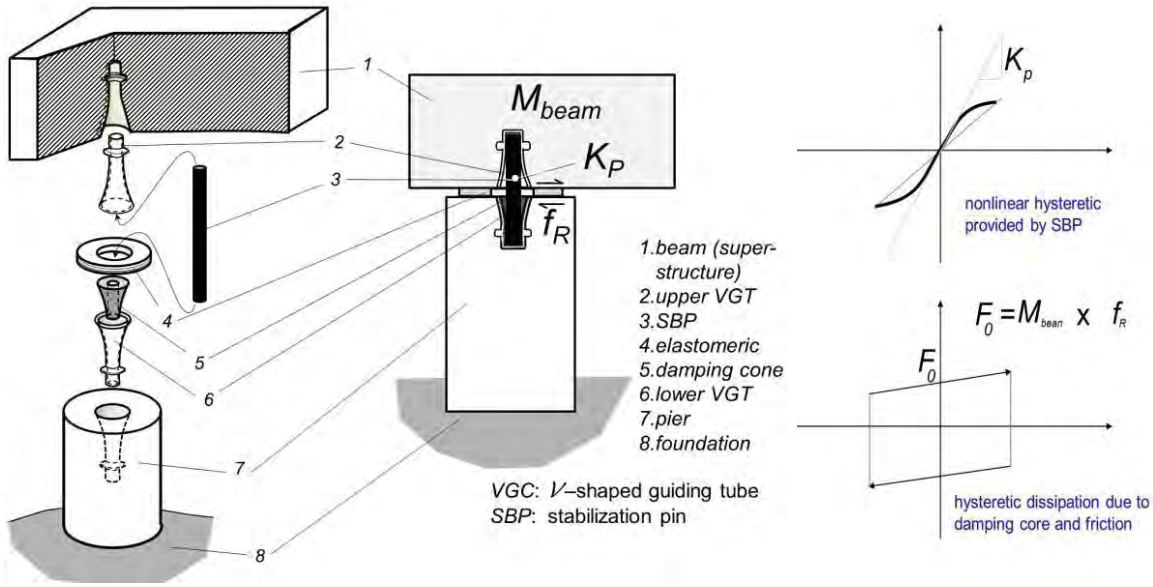

Figure 1. Concept of the V-Connector

MODEL AND ANALYSIS

On right of Fig. 2 is a physical model of a bridge structure with V-connectors, termed V-system in this article. On its left in the figure is the standard one-freedom model that is adopted to explain spectrum-based seismic resistant design in current industrial codes for bridges and buildings [2]. This model is more closed to CIP structure. As the comparison, the V-system on right is a two-freedom nonlinear dynamic model where the V-connector is presented by the

Kelvin-Voigt model that includes the Hooker's constant K_V to represent the stiffness of the pin (SBP) and the visco-plastic coefficient η_V to represent the dissipation cone; whereas u_p is the displacement of the pier top and u_d is the average deck's displacement above the pier. It is wellknown that a natural earthquake always radiates stress waves with distributed frequencies spectrum. For simplification, the dynamic behavior of the V-system are quantified by the group of two differential equations, presuming the system is struck by an external excitation in the form of single sinusoid wave with frequency ω and amplitude F:

$$\begin{cases} M_{deck}\frac{d^2u_d}{dt^2} + \eta_V\frac{du_d}{dt} - K_p u_p + K_V u_d = 0 \\ M_{pier}\frac{d^2u_d}{dt^2} + (K_p + K_V)u_p - K_V u_d = F \cdot sin(\omega t) \end{cases} \quad (1)$$

where t is time. To avoid tedious derivation, this proposal only presents the following displacement solutions that were derived after leaving out the visco-plastic term related to η_V.The solution, not loss generality, will provide basic information of the system's behaviors:

$$\begin{cases} u_p = \frac{[1-\bar{\omega}_d{}^2]u_{p0}}{U} \\ u_d = \frac{u_{p0}}{U} \end{cases} \quad (2)$$

where

$$U = [1 + \eta_M\bar{\omega}^2 - \bar{\omega}_p{}^2][1 - \bar{\omega}_d{}^2] - \eta_M\bar{\omega}^2 \quad (3)$$

and

$u_{p0} = F/M_{pier}$, $\bar{\omega} = \omega_d/\omega_p$, $\bar{\omega}_p = \omega/\omega_p$, $\bar{\omega}_d = \omega/\omega_d$, $\eta_M = M_{deck}/M_{pier}$, and
$\omega_p = \sqrt{K_{pier}/M_{pier}}$: the first order natural frequency of the pier;
$\omega_d = \sqrt{K_V/M_{deck}}$: the first order natural frequency of the deck with V-connector.

The physical meaning of (2) is fairly simple, for which the denominator U in (3) can be rewritten in the form as:

$$U = 1 - \bar{\omega}_p{}^2 - \eta_M\bar{\omega}^2\bar{\omega}_d{}^2 - \bar{\omega}_d{}^2 + \bar{\omega}_p{}^2\bar{\omega}_d{}^2 \quad (4)$$

Considering the case with very large stiffness K_v for the V-connector so it is similar to a rigid connection, which, as explained previously, is essentially the case of CIP. Because very larger

K_V leads to very smaller ϖ_d, therefore, under this situation, the denominator U in (2) can be approximated by

$$U \approx 1 - \bar{\omega}_p{}^2 \quad (5)$$

Accordingly, when $\omega \rightarrow \omega_p$, i.e. the earthquake frequency is approaching the system's natural frequency, $\varpi_p \rightarrow 1$, $U \rightarrow 0$, which is the condition that resonated vibration occurs for the CIP model on the left of Fig. 2. Hence, by design of the flexibility embedded within the V-connector, represented as a finite value of

the stiffness K_v in the V-system in figure 2, the first-order resonated vibration frequency can be shifted to a desired value since the zero point of the denominator U in (4) is determined by the coefficients associated with the V-connector.

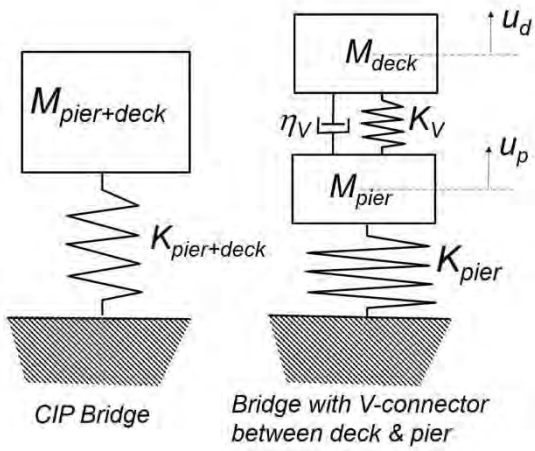

Fig. 2 The analytical model of the V-Connector and comparison with CIP bridge.

Potential for Bridges' Seismic Protection and Accelerated Construction

While earthquake is a continuing threat to the states with high seismic risks, Every Day Counts (EDC) in bridge's construction is a need in general for the country. A modern strategy to achieve accelerated bridge construction is "modular bridge", i.e. to build components off-site and then to assemble a bridge on-site, for which "assembling precision" and "connection strength" are the two remaining challenges in practices. This is because a bridge's component, e.g. a beam, is with the length from tens to hundreds feet; whereas a common connector, for example, a pin, is often with the dimension of inches, implying the precision requirement at the scale at the order of one tenth inch. It is no-trivial to manufacture a hundred foot-long beam with such a precision at specified locals; enforced-assembling for the cases without satisficed precision consumes extra time and labors' costs, plus resulted additional residual stress and detriment in robustness.

Therefore, for the situation that requires high strength at connections, e.g. in active seismic zone, the construction method termed CIP (cast-in-place) becomes the primary option for concrete bridges, which disables ABC while introduces significant extra cost due to the necessary form- work. Additionally, resonated vibration of an entire bridge may occur when it is with high pier or piers with uneven heights. This leads to the development of seismic isolation design, utilizing isolation bearings as the connectors between beam and pier. Being a flexible connection, an isolation bearing is able to shift a bridge's natural frequency when struck by earthquakes while reducing the transmission of inertia force flow. However,

such flexibility may also compromise the robustness needed for a bridge under some situations. For example, during the 2011 Sandai

Earthquake, numerous bridges were survived by the 8.9 degree earthquake but their superstructures were washed out by the following tsunami.

To this end, the developed V-connectors family aims at achieving the permanent robustness as CIP structure while possessing the temporal, restorable flexibility as that a conventional seismic isolation bearing has. Additionally, the V-geometry by such a connector enables modular bridge's fast on-site assembling with regular manufactural precision for bridge's component. The connecting mechanisms illustrated in Fig. 1 imply that the connector is also able to accommodate the temperature and live load-induced beam's deformation. In summary, the *payoff to practice* by applying the developed V-Connector products family can be briefed as the following properties:

-Sustainability and integrity: exceptional capability to protect a bridge from the damages caused by seismic events and by hurricane/tsunami while provide robust connection under regular conditions

-Efficiency: with the least requirement for maintenance during day to day operation, as compared with other kinds of bearings.

-Cost-effectiveness: competitive manufacturing cost, convenience for accelerated construction and replacement.

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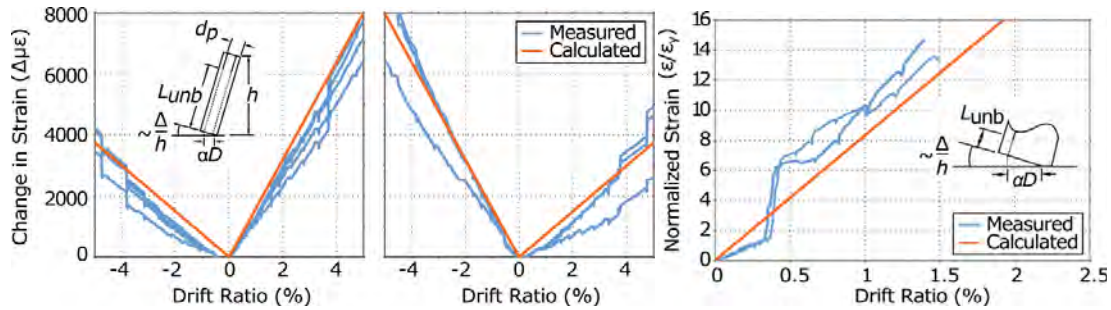
SHAKING TABLE PERFORMANCE OF A NEW PRECAST BRIDGE BENT SYSTEM WITH PRE-TENSIONED, ROCKING COLUMNS

Travis Thonstad[1], Islam Mantawy[2], John Stanton[3], Marc Eberhard[3], David Sanders[4]

A new precast bridge bent system has been developed to accelerate on-site construction, minimize residual displacements even after large seismic events and reduce post-earthquake damage. The system reduces on-site construction time by using prefabricated concrete components that are shipped to site and assembled there. The connections between these components are designed to provide generous construction tolerances and are both simple and quick to complete on-site. The system uses vertical, pre-tensioned strands in the columns that are bonded only at the column ends. The strands are designed to return the columns to vertical after the shaking has stopped. Specially detailed ends concentrate deformations to two large rotations at the precast connection interfaces and confine and protect the column concrete. The mild steel reinforcement crossing the connection interfaces is locally debonded.

The connections are the key to the system's seismic performance. They have been tested successfully under quasi-static conditions and found to perform exceptionally well, re-centering with very little concrete damage or residual drift after being loaded cyclically up to drift ratios of 10% [1]. Field experience with a similar, non-prestressed system suggests that the new system can be constructed rapidly [2].

The design of the new system involves several unique aspects. The initial prestress in the strands and the debonded length of the mild steel reinforcement are selected to prevent yielding of the strands and fracture of the reinforcement at the predefined design drift. Because the columns rock as nearly rigid bodies between the foundation and the cap beam, changes in strain in the prestressing strands and the mild steel reinforcement can be approximated using rigid body kinematics, as shown in Fig. 1.

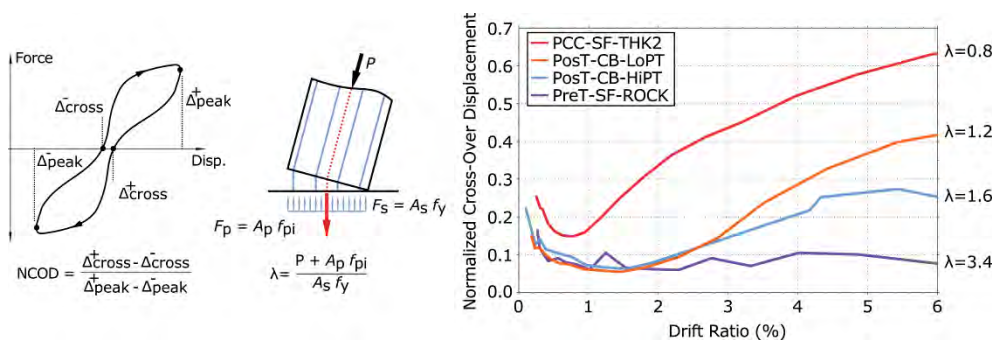


(a) *(b)*

Fig. 1: *Envelopes of changes in strain for column reinforcement in the subassembly connection tests [1], (a) prestressing strands, (b) mild steel reinforcing.*

The mild steel reinforcement and prestressing strand areas are proportioned to encourage re-centering. The desired level of re-centering, and thus the amount of residual drift at a given maximum drift, can be quantified using the Normalized Cross-Over Displacement (NCOD) and the dimensionless re-centering parameter, λ, defined in Fig. 2(a) and 2(b) respectively. The NCOD-drift ratio relationships for several subassembly connection tests are shown in Fig. 2(c).

For columns with λ values greater than unity, the measured NCOD values are greatly reduced from the reference precast, non-prestressed column (λ=0.8). Furthermore, the column with λ=3.4, performed better (lower NCOD values) than those with smaller re-centering ratios at higher drift ratios. These results suggest that the system can be designed with nearly any desired level of re-centering.



(a) *(b)* *(c)*

Fig. 2: *Re-centering performance of subassembly tests, (a) definition of the NCOD, (b) definition of dimensionless re-centering ratio, λ, (c) NCOD versus drift ratio for reinforced concrete [3] and prestressed column specimens [1,4].*

The dynamic performance of the new system was measured through multi-shaking table tests conducted on a quarter-scale, two-span version of the bridge, shown in Fig. 3. The specimen was subjected to a series of increasingly severe ground motions. The geometry and strength of the bridge and some of the

experimental motions were chosen to match those of a previously tested bridge, constructed using conventional cast-in-place detailing [5].

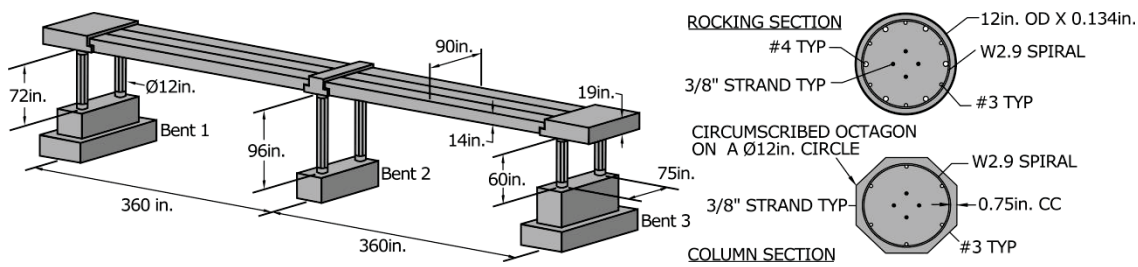


Fig. 3: *Shaking table specimen dimensions and critical sections.*

The new system behaved as intended through the design earthquake, sustaining only cosmetic damage and exhibiting excellent re-centering properties. The maximum and residual drifts for the shortest bent (Bent 3) of the pre-tensioned and reinforced concrete bridge specimens are shown in Fig. 4. Up to the design level earthquake the maximum drift ratios in the two systems was similar. Beyond the design level motion the drift ratios were higher in the pre-tensioned system; however the residual drift ratios were lower on average. The maximum residual drift ratio during testing was 0.4%, even after excursions to drift ratios exceeding 13% [6].

At the end of testing (up to 220% of the design level), many of the reinforcing bars had fractured but there was essentially no concrete damage, as shown in Fig. 5(a). Only three residual flexural cracks were observed during testing, none exceeding hairline in width. Rebar fracture, first occurring during the 133% Design Level motion (with a maximum drift ratio of 5.7% in Bent 1), could be delayed by additional debonding of the longitudinal reinforcement. This is in contrast to the reinforced concrete bridge which sustained significant spalling, spiral fracture and bar buckling during the 220% design earthquake, as shown in Fig 5(b).

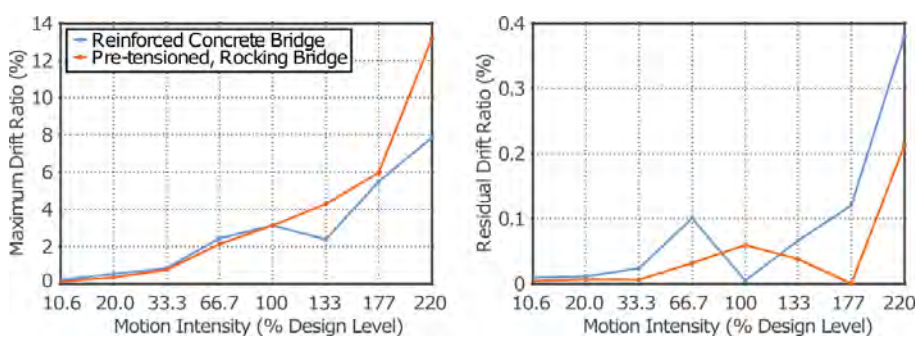


Fig. 4: *Maximum and residual drifts for Bent 3 for the high-amplitude coherent motions.*

(a) *(b)*

Fig. 5: *Damage to the column concrete after the 220% design level motion. (a) pre-tensioned, rocking bridge, (b) reinforced concrete bridge.*

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G-8: ABC PROJECT USING SPMT

SPMT FOR ACCELERATED BRIDGE CONSTRUCTION – HIGHGROVE UNDERPASS

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INTRODUCTION

California Department of Transportation (Caltrans), in cooperation with San Bernardino Associated Governments (SANBAG) and Riverside County Transportation Commission (RCTC), proposed to widen a portion of the Interstate Route 215 (I-215) freeway for approximately 7.5 miles to include high-occupancy vehicle (HOV) lanes in the median. The proposed improvements would complete a regional HOV network to provide continuous HOV lanes from San Bernardino to Los Angeles along the I-215/SR-91 corridor.

The structure scope of work included the widening of several highway bridge on the I-215 mainline and also the replacement of a railroad bridge – Highgrove Underpass, owned by Burlington Northern Santa Fe Railroad (BNSF), due to a limited roadway width under the bridge. The existing railroad bridge is a steel through girder bridge carrying two railroad tracks. Currently, the roadway width underneath the bridge can only accommodate three traffic lanes in each direction. In order to accommodate the HOV lanes near the median, bridge piers would have to be relocated between traffic lanes. Also, the existing 2 column bent is inadequate for the increased design loads associated with longer spans. Therefore, it was determined that the existing bridge has to be removed and replaced with a longer span bridge. Since BNSF is the bridge owner, the design and construction of the replacement bridge needs to follow the American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual of Railway Engineering guidelines. Furthermore, design and construction strategies and methods of the new bridge must keep the railroad lines fully operational during construction.

BRIDGE TYPE SELECTION

The existing railroad bridge is a four-span through plate girder bridge that carried two sets of railroad tracks. Three south-bound and three north-bound highway lanes provide traveled way beneath Span 2 and Span 3, respectively. The fixed railroad alignment traverses the I-215 alignment at a 55-degree skew. The

railroad bridge span lengths are: 47.75 *ft* –103.67 *ft* – 122.00 *ft* – 54.75 *ft*. The out-to-out width of the bridge steel flange plates is 36.0 *ft*. Within the current project, one HOV lane will be added in each direction and the ultimate planned traffic layout will be five general purpose lanes plus one HOV lane in each direction. The task for bridge design is to accommodate the ultimate planned traffic layout.

The bridge type selected was a steel through truss structure. Several span configurations were proposed and evaluated during the structure type selection phase. After weighing various advantages and disadvantages of the span configuration alternatives, twin two-span steel through truss bridges were selected as the final structure type and each truss bridge will carry one railroad track. The twin bridges consist of two spans with each span length of 200 *ft*, for a total bridge length 406 *ft*. Each truss bridge width is 21 *ft*, and the structure through truss height is *30* ft. The two identical twin Warren truss superstructure will be simply-supported on the new piers. Based on the narrower single track bridge width and a compromised median width at Bent 2, the skew is reduced to 20-degrees. The advantages of this alternative include a reasonable span length for easy construction, and a reasonable skew for reducing live load fatigue and seismic effects.

BRIDGE CONSTRUCTION

Based on the original design, the construction phases for the bridge replacement consists of the following steps:

1). Construct temporary foundations and piers at the shoofly location and build a temporary steel through plate girder shoofly structure

2). Reroute the two mainline railroad tracks onto the shoofly structure

3). Demolish the existing Highgrove Underpass

4). Construct foundation and piers at the permanent underpass locations and build three new Highgrove Underpasses (two-span trusses)

5). Shift the two mainline railroad tracks back onto the new structures

6). Demolish shoofly structure.

At the beginning of construction, the contractor found that the floor beams for the steel through truss girder shoofly structure would not be available for at least six months. In order to avoid construction schedule and project delays, the contractor proposed a revised structure construction strategy and which

was reviewed and accepted by Caltrans. The proposed strategy uses Self Propelled Modular Transports (SPMT) technology to achieve accelerated bridge construction (ABC) and consists of the following phases:

1). Construct temporary foundations and piers at the shoofly locations

2). Build Truss #1 (permanent truss) offsite and use SPMT to move the truss onto the Shoofly #1 location

3). Build Truss #2 (permanent truss) offsite and use SPMT to move the truss onto the Shoofly #2 location

4). Reroute Mainline Track #1 onto Truss #1 and Mainline Track #2 onto Truss #2 at their shoofly locations

5). Demolish the existing underpass bridge

6). Construct new foundations and piers at the original Mainline Track #1 and #2 and the future Mainline Track #3 locations

7). Built Truss #3 and use SPMT to move the truss onto the future track location and then reroute Mainline Track #1 onto Truss #3

8). Use SPMT to move Truss #1 onto the permanent Mainline Track #1 location and then reroute Mainline Track #2 onto Truss #1

9). Use SPMT to move Truss #2 onto the permanent Mainline Track #2 location

10). Shift Track #2 back onto Truss #2

11). Shift Track #1 back onto Truss #1

12). Demolish temporary foundations and piers at the shoofly locations

The contractor's phasing strategy accommodates Caltrans closures and traffic control schedule as previously planned and also presents the following advantages:

- Eliminates extensive erection of the temporary shoofly structure and the subsequent demolition directly adjacent to live railroad tracks

- Reduces the original two demolition operation phase down to one phase.
- Reduces traffic exposure and safety hazards to the traveling public
- Reduces interface with and disruption to railroad operations and the potential for track fouling
- Reduces the operations cost for railroad flagging personnel

Extended freeway closures were secured by Caltrans, and BNSF agreed to provide extended track closures up to 20 hours to switch tracks. The use of SPMT allowed the steel fabricator, Stinger Bridge and Iron, to begin assembling the trusses at the bridge farm, located approximately 0.75 miles from the permanent bridge locations, while work began simultaneously on the substructure (foundations, piers, and abutments) for the shoofly bridges.

Truss assembly and substructure work initiated in February 2014 and the substructure work along with the truss assembly for the two shoofly bridges was completed at the beginning of April 2014. During the remainder of the weekends in April, the two permanent trusses were transported by SPMTs to the shoofly locations. By June 2014, BNSF completed realignment of the tracks to the new shoofly bridges. Demolition of the existing underpass bridge was completed by the end of July 2014. Next, work on the substructure for the 3 permanent truss bridges began while the trusses for the 3rd bridge were being assembled. By the end of August, the substructure work for the permanent truss bridges was completed. The two trusses completed the shift by SPMT from the shoofly to their permanent locations by September 20th, 2014. The entire process of building the shoofly, switching the railroad tracks, demolishing the existing bridge, and building the new permanent bridges took approximately 8 months. The contractor estimated approximately 20 months for construction without SPMT, and the material delay would have added another 12 months due to the 4th quarter railroad shutdowns. By using SPMT construction process, the overall cost saving to the State of California is approximately $400,000, and the overall time saving is approximately 20 months.

CONCLUSION

The Highgrove Underpass project exhibited the successful implementation of SPMT technology to achieve accelerated bridge construction which has significantly reduced traffic exposure and safety hazards to the traveling public and workers, reduced potential risk of disruption to railroad operations and commerce, and accelerated the project delivery schedule. The project also underscored the close partnership between the state agency and the contractor and the success of SPMT to achieve ABC rests upon the collaboration between the two parties.

The days of blazing new highways in virgin territory are largely past. Today's transportation officials have become more cognizant of the impacts their planned works have on traffic and have been proactively searching for solutions to minimize such disruptions. Political pressures manifested through regional funding leveraging have solidified the above as a basic parameter in the transportation improvement equation across the nation and throughout much of the civilized world. Thus, it is expected for planners and engineers to plan and formulate accelerated bridge construction solutions such as SPMT and/or prefabricated bridge elements to address demands to accelerate construction and delivery schedule and reduce on-site impacts to the travelling public and existing commerce.

Rapid bridge construction has been a leading focus of national and international research, with numerous funded projects in academia and industry. Recently in the United States, the federal government and many states including California have focused on implementing ABC technologies that have been formulated and developed from past completed research. The need for accelerating transportation construction technologies is increasing in North America, European, and Asian countries and the widespread implementation of these technologies will benefit commerce and the traveling public, leading to a "win-win" scenario.

THE 123'S OF SPMT'S FOR ABC

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ABSTRACT

In the absence of technical information being readily available on the subject matter, this paper presents the ABC user details on how Self-Propelled Modular Transporters work by presenting specifications of a common SPMT manufacturer [Goldhofer], and examining the limits states that affect the net capacity.

In particular, an in depth discussion of structural and hydraulic capacity is presented to help the reader understand how a stability area is established through hydraulic zoning of an SPMT and how the loading or travel configuration may limit the overall capacity.

INTRODUCTION

Self-Propelled Modular Transporters [SPMTs] are computer-controlled platform transporters with hydraulic suspensions, capable of moving several thousand tons. An SPMT is propelled by hydraulic drive motors which are mounted on the axle. A diesel driven power pack mounted at either end of the SPMT houses a hydraulic pump and provides power to the various hydraulic functions of the SPMT. When operated properly, they may be controlled within centimeters.

SPMTs were developed in Europe, where they have been used extensively for many years. Up until recently, the use of SPMTs in the United States was mainly limited to the Heavy-Civil and Industrial markets and typically used to transport heavy machinery. More recently, SPMTs have been employed as a method of delivery for Accelerated Bridge Construction [ABC]. The use of SPMTs in ABC allows construction of new bridge structures to be built offline and under controlled conditions with minimal disruption to traffic and the public, then driven into place during an abbreviated road closure.

SPMT FUNDAMENTALS

The term SPMT is generic – many brands are available and readily used throughout the US. Each manufacturer has its own specific geometry and specifications, but all share one commonality – hydraulic suspension of the transporter axle lines, which are capable of being linked together to form suspension groups or zones to provide stability. Goldhofer brand transporters are discussed at length in this paper due

to the authors' familiarity, however other brands function similarly. Since SPMT's were mainly developed in Europe, SI Units are typically given to define their specifications.

Bogie

A bogie consists of the spine beam which carries the load on the platform to a pair of wheelsets which make up each axle line; Goldhofer transporters are comprised of one or several bogies. Each bogie is 3000mm [9.84ft] wide and defined by the number of axle lines. Goldhofer offers bogies of 2, 3, 4, 5 and 6

Figure 1: Bottom View - 6-Line Bogie

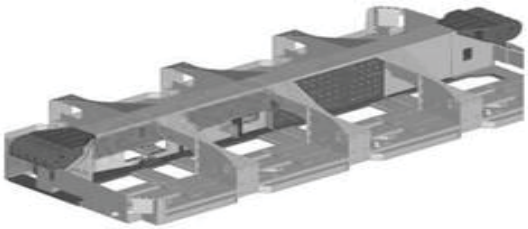


Figure 2: Bottom View - 4-Line Bogie [wheelsets removed]

axle lines, while the 4-line and 6-line are most prevalent. Axle line spacing is 1500mm [4.92ft], which makes the bogie length equal to the number of axle lines multiplied by 1500mm. Several bogies may be coupled together in both the longitudinal and transverse [double-wide, 1.5 wide] directions to offer endless transporter combinations.

Each bogie contains: integrated hydraulic plumbing to control the hydraulic suspension and wheelset drive motors, air lines for the braking system, and a steering assembly. The steering assembly may be either mechanical or electronic and is covered in detail later in this paper.

Wheelset

Each wheelset is comprised of a pendulum axle supported by a hydraulic cylinder mounted to a turn table

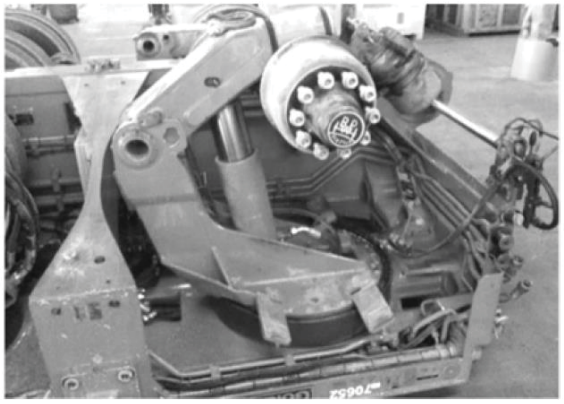


Figure 3: Bottom View - Wheelset w/ Tires Removed

Figure 4: Goldhofer traversing uneven terrain

on the underside of the platform. This is the hydraulic cylinder that will be linked together with other similar hydraulic cylinders of adjacent wheelsets to form a hydraulic zone as previously noted. The wheelset is mounted to the lower leg of the pendulum axle and free to cope with road camber and uneven terrain. The hydraulic cylinder extends or retracts either under influence from the terrain or control of the hydraulic pump [levelling of the deck] by the operator. Each axle line consists of two independent wheelsets, one each on either side of the spine beam. Furthermore, each wheelset has [4] tires – [2] each on either side of the pendulum [8 total per axle line].

Figure 5: Driven Axle [Left] and Non-Driven Axle [Right]

Typically, not all wheelsets on a bogie are driven. Driven wheelsets are equipped with hydraulic drive motors mounted in the hubs of the axles [See Figure 5].

STEERING

As noted in the discussion above, each wheelset is mounted to a turntable at the underside of the platform deck and may be controlled either mechanically or electronically. Mechanical steering refers to the presence of steering rods which physically connect every axle to an adjacent axle. The available steering angle of a mechanical steering arrangement is limited by the physical restrictions of the steering rods. In general, these arrangements may be operated ±50° from the longitudinal transporter axis. Counter Steer and Front Steer, as shown in Figures 6 and 7 below, highlight the typical maneuverability of a mechanical steering arrangement. Operating in Counter Steer allows a turning radius one-half of the size of the same transporter operating in Front Steer.

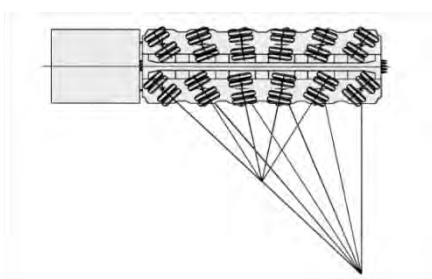


Figure 6: Mechanical Steering - Counter Steer

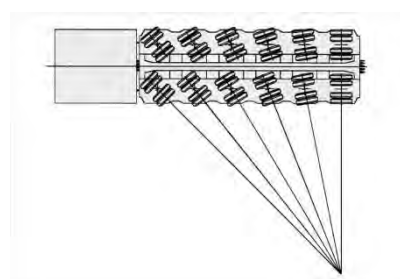


Figure 7: Mechanical Steering - Front Steer

On the other hand, there is no physical connection between adjacent wheelsets for an electronic steering arrangement [PSTe]. Rather, the turntable of each wheelset is equipped with its own means of operation –

typically a worm gear or hydraulic cylinder. A signal from the Operator's control box is sent to the CPU of the SPMT and dictates the appropriate steering angle for each wheelset. Since there is no physical limit

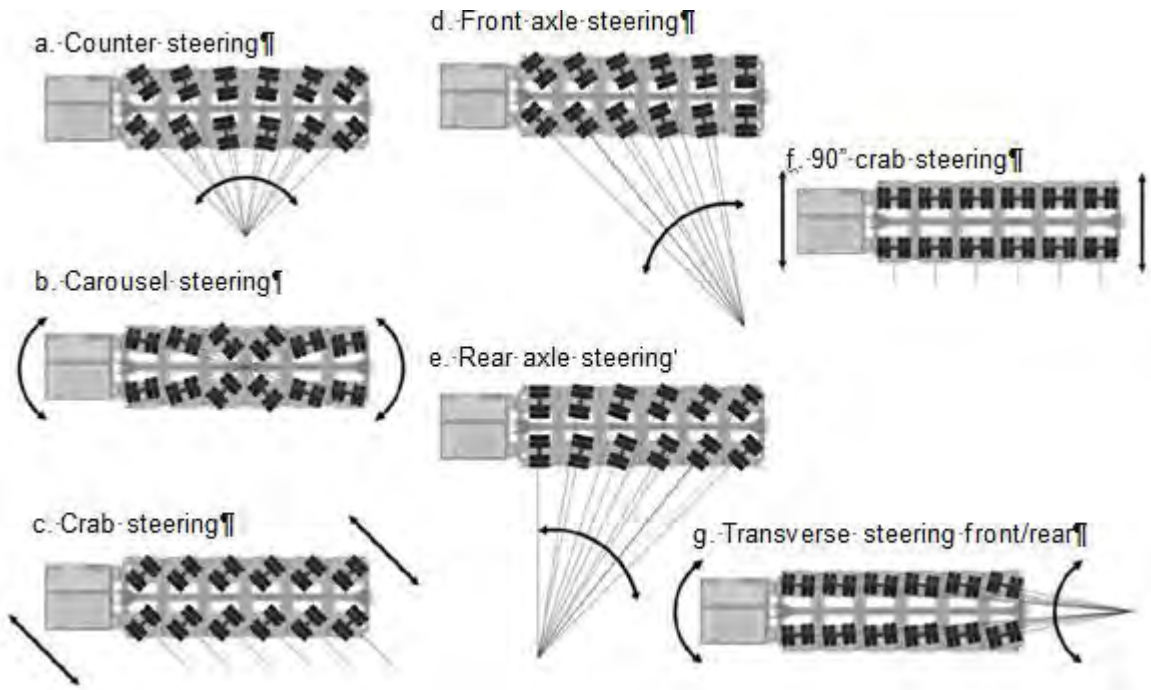


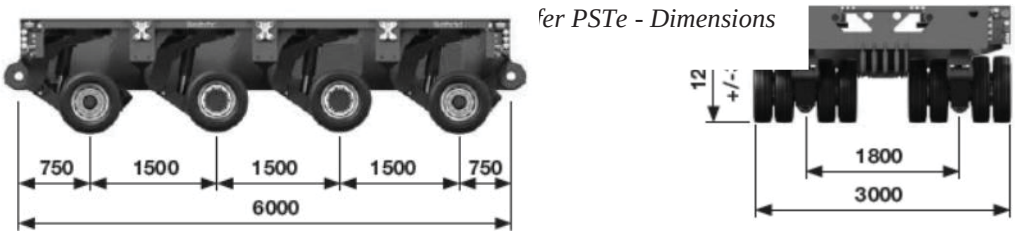
Figure 8: *Electronic Steering Modes Available*

on the steering angle, electronic steering is able to provide unlimited maneuverability. Available steering modes for electronic steering configurations are illustrated in Figure 8 below:

TECHNICAL DATA

Overall Dimensions

As previously noted, a typical 4 axle bogie is 3000mm [9.84ft] wide and 6000mm [19.7ft] long [4 axle lines x 1500mm axle spacing = 6000mm]. The length of the transporter is measured from center to center of coupling shaft. Thus, as additional bogies are added, the axle spacing remains constant.



er PSTe - Dimensions

The median deck height for Goldhofer PSTe is 1220mm [48in], however the deck height may vary +/- 300mm [11.8in] due to the available hydraulic stroke. Each tire is 215mm [8.5in] wide on 17.5in rims and inflated to 8.5bar [120psi]. It should be noted, in consideration of the minimum deck height, 920mm [36.2in], the structural components of the transporter bogie will be in contact with the tires of each axle line which would prevent movement.

The PSTe powerpack may be mounted on either end of the transporter, though it is preferable it be mounted on the back or opposite the direction of travel. The powerpack is the same width as the standard bogie and measures 3975mm [13.0ft] long. The reservoir of a single PSTe powerpack may control up to 24 axle lines. Additionally, the height of the powerpack may be adjusted ±50° from horizontal.

CAPACITY

The capacity of an SPMT transporter is often a much ballyhooed topic for any given ABC project. In general, there is both a structural and hydraulic capacity that must be considered for any transporter. In addition, the stability of the transporter [which may or may not affect hydraulic capacity], must be analyzed. Finally, the ground capacity must be investigated. To consider the capacity of any SPMT transporter, both the transporter and loading configuration must first be known.

Structural Capacity

Let us consider first the structural capacity. A check of the transporter spine beam is warranted to assure the loading condition may be safely supported. Transporter configuration, number of axle lines and payload support locations all factor into the check of spine beam capacity. In general, this is a beam check on an elastic foundation. A transporter is typically loaded at no less than two load points along the spine beam.

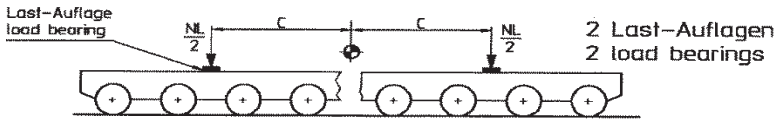


Figure 10: *Load Support Dimensions [Goldhofer]*

Goldhofer provides loading diagrams to simplify this check. In Figure 11 below, a load diagram for an 18-Line PST Goldhofer SPMT is shown. If two load points were considered to support the payload, the

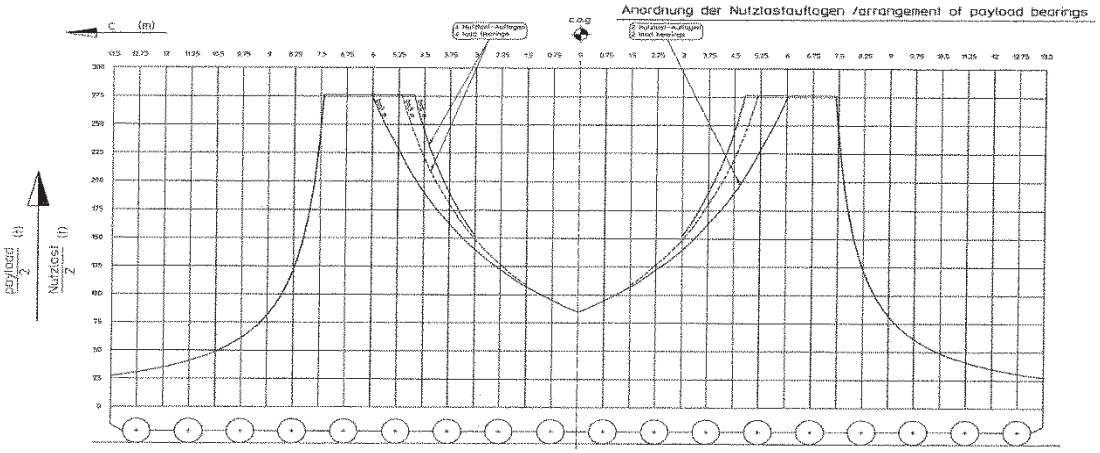


Figure 11: *18-Line PST Goldhofer Spine Loading Diagram*

optimum location for these two supports would be between 6.0m - 7.5m on either side of the transporter CG. As noted on the y-axis of the loading diagram, the payload capacity [based on structural capacity alone], is 550tonne or 275tonne per load application point. Note that as you move either side of these support locations, the structural capacity of the transporter drastically decreases.

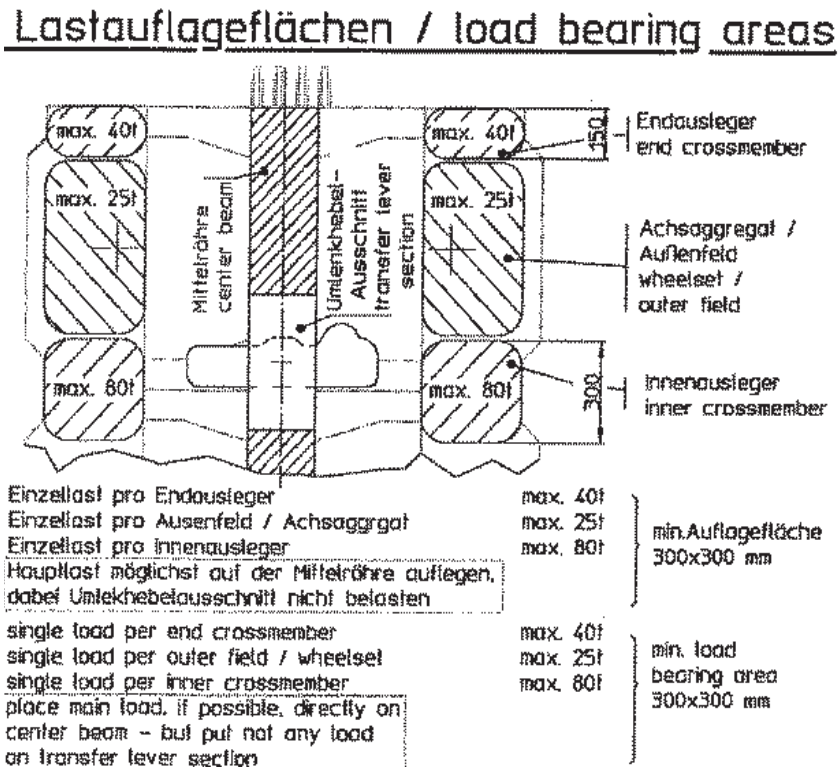


Figure 12: Goldhofer Deck Loading Diagram

In addition to spine beam capacity, the structural capacity of the transporter deck must also be verified. Goldhofer also provides a loading diagram of appropriate deck loading locations [See Figure 12]. If neither the spine beam nor deck loading constraints may be met, additional grillage or other load spreading material must be added to the transport configuration.

Hydraulic Capacity and Stability

The hydraulic capacity of an SPMT is simply the sum of the capacities for each hydraulic cylinder of the wheelsets on the transporter. All axles, whether drive axles or not, have the same capacity however the travel speed of an SPMT transporter may reduce the hydraulic capacity. As the travel speed increases, the hydraulic capacity of each axle is reduced to account for inertial effects on the payload. For the purposes of ABC, travel of the SPMT is typically limited to less than a walking pace, thus the maximum hydraulic capacity may be assumed. A PSTe operating under ideal conditions has a maximum hydraulic capacity of 36tonne [39.6T]. Payload capacity refers to the allowable load per axle [pair of wheelsets] minus the deadweight of the transporter.

As previously noted, the hydraulic suspension of an SPMT allows several wheelsets to be linked together to form suspension groups or zones. These zones create a hydraulic platform or stability area – this is

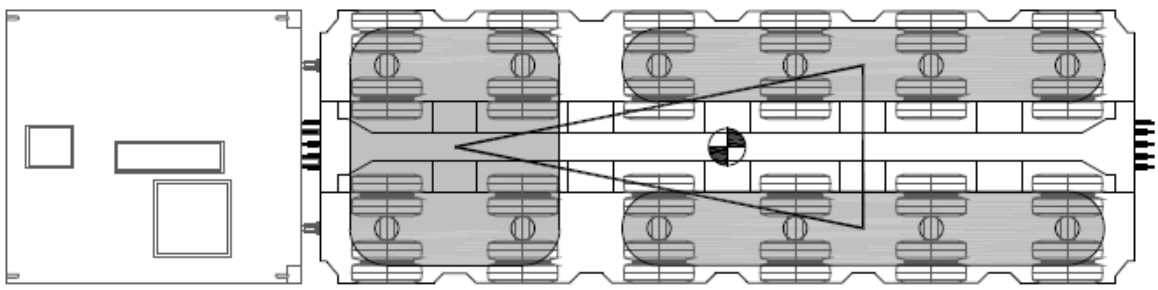
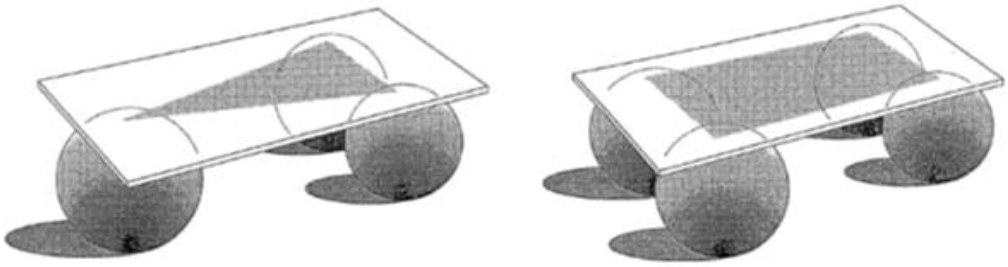


Figure 13: Layout of 3-point zone for 6-Line Goldhofer

often referred to as zoning the SPMT. The center of each hydraulic suspension zone creates a corner of the stability area. Given the pendulum axles of SPMT transporters, the stability area is considered to be located at the fulcrum of the pendulum. An SPMT may be zoned as either a 3-point suspension or 4-point suspension. While a 4-point zone provides a greater stability area, it is susceptible to hydraulic overload. If the center of each hydraulic zone is dramatized as a beach ball, one may consider that a plate supported on [4] beach balls may not necessarily sit level and could be prone to rocking from one corner to the opposite corner. If the plate in this arrangement were to rock, the plate would be effectively supported by only [2] beach balls, or hydraulic zones. Depending on the magnitude of the load on the plate, this may



result in failure of one of the beach balls, or in the case of an SPMT transporter – failure of a hydraulic zone due to overload. Alternatively, the same plate supported on [3] beach balls, or hydraulic zones, will not rock. Instead, the plate may move out of level to compensate for uneven support of or un-level ground conditions. It is for this reason that a 3-point zone is almost universally specified for SPMT operations including ABC. As long as the combined center of gravity [payload and transporter], remains within the stability triangle, the transporter may be considered hydraulically stable.

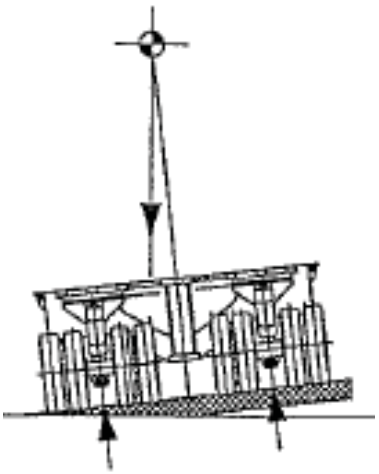
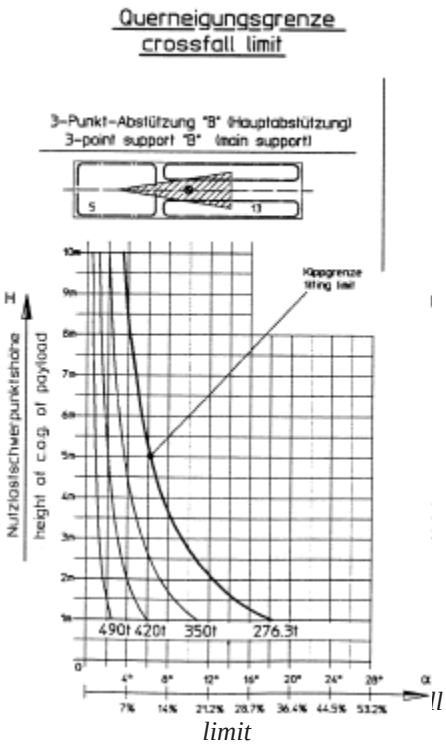


Figure 15: Crossfall

As the SPMT travels over uneven ground, cambers or changing gradient on the travel path, hydraulic fluid in each suspension zone will free flow to maintain equal loading to each axle or wheelset in the suspension group. Alternatively, the hydraulic fluid may be pumped into any of the hydraulic zones in order to level the deck. As the deck of the transporter tilts in response to travel or levelling, the CG theoretically moves toward the boundary area of the stability triangle – this is referred to as *crossfall*. The angle of the transporter deck that would result in the CG reaching the edge of the hydraulic stability is referred to as the *tilting limit* for an SPMT. The tilting limit is independent of the payload weight and influenced only by the CG height and zoning geometry. An overload limit may also be determined using similar theory. As the CG moves within the stability area, two zones of the 3-point suspension may see increased loads. The angle of the transporter deck that would result in sufficient CG movement to influence hydraulic overload of a suspension zone is referred to as the *overload limit* for an SPMT. For standard transporter configurations, Goldhofer provides a chart to determine the tilting and overload limits as a function of CG height [tilting] and payload weight [overload]. For more complex transport configurations, like those typically found in ABC, the geometry of the hydraulic suspension zones may be solved graphically and the crossfall limit determined using trigonometry.

In addition to the stability factors already discussed, other external factors may also negatively affect the hydraulic capacity of an SPMT. Acceleration/deceleration from the start and stop of the transporter, wind on the payload and other dynamic forces related to the travel path all must be considered when analyzing hydraulic capacity. In the absence of detailed analysis, it is prudent to apply



limit

a 10-20% reduction to the stated maximum payload capacity; an allowable payload capacity of 30tonne [33T] is conservatively considered to be the hydraulic capacity per axle for a Goldhofer PSTe.

Ground Bearing Pressure

It would be remiss to discuss the capacity of an SPMT transporter without a discussion of the travel path or ground supporting the transporter. For the ground loading beneath an SPMT, there are two cases to consider: Tire Contact Pressure and Pier Load. The tire contact pressure is simply the tire inflation pressure – this is true for any vehicle regardless of size and/or weight. This value may be useful for long-term loading, however is largely irrelevant for heavy transport and moving loads. The pier load or *shadow pressure* is a more accurate representation of the ground loading condition below the transport during SPMT operations. The shadow pressure is defined as the gross load of the laden transporter

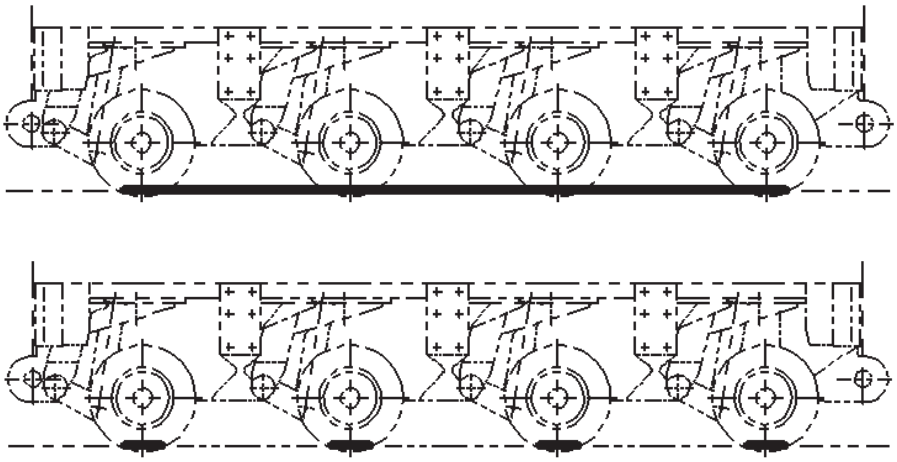


Figure 17: *Goldhofer illustration showing Tire Contact Pressure Area [top] vs. Pier Load Area [bottom]*

divided by the *shadow area* of the transporter – or the area measured taking the distance from the front to rear axle and multiplying by the transporter width. The capacity of the soil and/or structures will obviously vary site to site and a qualified engineer must determine the capacity correctly for a transient temporary load in order for the SPMT operation to be succesful.

SUMMARY

In summary, the continued use of Self-Propelled Modular Transporters in Accelerated Bridge Construction warrants the need for a better understanding of how they operate and how the loading or travel configuration may inhibit the overall capacity. Linking the hydraulic suspension of an SPMT to establish hydraulic zones and create a hydraulic platform provides stability to the load on the platform deck. As the SPMT travels over changing ground conditions, the free flow of the hydraulic fluid through the suspension group maintains equal axle loading in that suspension group.

In a perfect world, the combined center of gravity of the payload and self-weight of the transporter would align with the center of hydraulic support and center of stability area during the entirety of a heavy haul or ABC bridge move. If this were the case, no reduction in the maximum hydraulic capacity would be necessary and tilting or hydraulic overload would not be a concern. However, SPMTs are never used in these idealized conditions, thus external forces and out-of-level conditions must be assumed, analyzed and mitigated to the extent possible through Professional Engineering.

To hearken back to Archimedes, “Give me space to travel, and a center of gravity, and I could move the world on an SPMT.”

REHABILITATION OF THE PARK AVENUE AND WATCHUNG AVENUE BRIDGES FOR NEW JERSEY TRANSIT USING SELF-PROPELLED MODULAR TRANSPORTERS

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INTRODUCTION

The Park and Watchung Avenue Bridges each carry New Jersey Transit's Raritan Valley Line (RVL), an extremely busy commuter rail line, over two congested local streets in downtown Plainfield, New Jersey. The structures are located adjacent to Plainfield Train Station, with one bridge at each end of the station's high-level platforms. Additionally, the bridges pass through a very busy urban area with many local businesses and restaurants. Given the age and condition of the superstructures and substructure elements, New Jersey Transit (NJ Transit) embarked on a major rehabilitation project for both bridges as part of their ongoing Capital Program. This project included replacement of the superstructures, as well as miscellaneous substructure repairs. The existing superstructures were in a state of major disrepair due to over-height vehicle impact damage and general deterioration and corrosion due to age. The method of Accelerated Bridge Construction chosen for the project was the use Self-Propelled Modular Transporters (SPMTs). The use of SPMTs allowed for the bridges to be built at a remote offsite staging yard and rolled-into place, thereby reducing the duration of local road closures at the bridge sites during construction and minimizing the impacts to local businesses in the immediate vicinity.

Both bridges support two tracks and are three-span, simply supported ballasted deck structures. Each structure has two approach spans over sidewalks along both sides of the street (16'-3" maximum span) and a main span over the roadway (35'-4" maximum span). While each structure was originally constructed to carry four tracks, only the inner bays carry tracks today. The fascia bays are utilized for access along the right-of-way by NJ Transit forces and to carry buried signal and communication cables. The existing spans were comprised of riveted steel built-up through girders with rolled steel floorbeams and a steel deck plate retaining the ballast. They are supported on full height masonry abutments and latticed multi-column steel pier bents located on the sidewalk along the roadway curbs. The existing superstructure and pier bents of the Watchung and Park Avenue bridges were built circa 1904, however the abutments date back to earlier construction prior to past superstructure replacements.

DESIGN

As these bridges are both part of the Central Railroad of New Jersey Main Line Corridor Historic District, the project required review and approval by the State Office of Historic Preservation (SHPO). In order to preserve the historic character defining features of the existing bridges, the proposed superstructures were designed to retain the original riveted steel built-up through girder framing, as well as the three-span configuration. Additionally, the fascia girders of each bridge were removed, taken offsite to be rehabilitated and later reattached to the new superstructure after the interior track bays were rolled into place. As another means of preserving the historic nature of the bridges, high strength bolts with rounded heads were utilized to mimic the existing rivets at connections visible to the public.

The existing vertical underclearance of both bridges was also substandard. This had resulted in numerous impacts to the existing fascia girders by over-height vehicles throughout the years. However, given that the existing Plainfield Station high-level platforms are located directly adjacent to the existing bridges, the profile of the existing tracks had to remain unchanged. Coincidentally, past track maintenance on the RVL had led to past profile increases and excessive ballast depth on the bridge. Therefore, additional underclearance was gained by this project through a reduction of this ballast depth to NJ Transit's minimum acceptable limit of 8". This raise in profile, however, triggered the need for modifications to the existing steel pier bents, abutment seats and backwalls to accept the raised superstructure.

ABC CONSTRUCTION METHODOLOGY AND STAGING

As previously discussed, the main objective of this project was to complete the superstructure replacement and substructure repairs with minimal impact to rail operations and the surrounding community. As the RVL was reduced from four tracks to two tracks during the 1980s, the majority of the bridge superstructure replacements had previously been accomplished through the shifting of the active tracks to the unused fascia bays of the bridges. This staging strategy had proven effective; however, the proximity of the Watchung and Park Avenue Bridges to the high-level platforms at Plainfield Station eliminated this staging methodology for these two structures.

The possibility of utilizing long-term single-track outages for replacement of one bay at a time was quickly dismissed given the long distance of over 7 miles between interlockings on the RVL. This distance would make single tracking impractical during weekday peak periods given the current service levels. Additionally, a one-track outage and a staged superstructure replacement was further complicated by the fact that the middle girder is a common support element for both tracks and is required during both stages of construction.

The chosen alternative for the project was to utilize a two-track outage over a long weekend beginning Friday night at 9pm and ending on Monday morning at 6:00am with full service restored for the morning commute on both tracks. In order to accomplish the replacement within the tight 57-hour construction window, it was determined that a roll-out/roll-in operation for the main roadway span would be most viable. However, the use of a traditional lateral slide-in and slide-out on temporary bents constructed in the roadway adjacent to the existing bridge on each side was not desirable, as both bridges are on key arterial roadways within the downtown business district. The area required for construction of the proposed bridge and demolition of the existing could not be accommodated without lengthy road closures with major impacts to these businesses and disruptions to Plainfield Station pedestrian access.

Conversely, the use of SPMTs allowed for the bridges to be completely erected at a remote offsite staging yard and rolled-into place during the weekend two-track outage. This reduced the length of time required for local road closures during construction. Alternating short-term closures of Watchung and Park Avenue were still required in order to perform repair work on the abutments and pier bents, as well as to prepare the substructure elements to accept the replacement superstructure, but these closures were significantly shorter in duration than would have been needed for the temporary bent alternative.

The selection of the offsite staging area and the SPMT roll-in/roll-out routes were carefully coordinated with officials from the City of Plainfield and Union County. Consideration was given to the proposed SPMT route lengths, the MPT required for the bridge moves and the associated local traffic detours. Horizontal and vertical clearances along each possible route were surveyed and checked to determine that there were no interferences with building or other structures. All affected public and private utilities owners were consulted to determine where interferences would occur and whether these utilities could be easily relocated during the bridge moves. It was ultimately decided to utilize a local side street within a few blocks of each bridge. During construction, the Contractor submitted detailed SPMT work plans outlining the proposed operation, including verification that the roll-out/roll-in route was cleared of obstructions or interferences and design checks of the SPMT loads on the existing roadway pavement. CAD drawings of the proposed route illustrated the arrangement and path of the SPMTs and bridge structures along the travel routes to and from bridge sites and the offsite construction staging location. Although the original design plan called for all five through girders of the new roadway span superstructure (two rehabilitated fascia girders and three interior girders) to be rolled in together, the Contractor requested to roll-in only the three interior girders, instead choosing to erect the fascia girders by traditional methods outside of the weekend two-track outage. This made maneuvering the SPMT unit through the local streets easier and reduced the number of direct conflicts with interferences such as tree or utilities.

As of this writing and due to unforeseen delays on the project, the SPMT roll-out/roll-in operations have not yet been performed, with the first move currently scheduled for the weekend of October 30 to November 1, 2016. However, a detailed construction sequence and schedule for the weekend operations has been established through coordination between NJ Transit, HNTB and the Contractor. Once the weekend two-track outage is in place at the first bridge, NJ Transit forces will work to remove the existing rails and ties. After completion of this effort, the contractor will remove the existing ballast from the bridge structure. The sidewalk end spans will then be lifted out by crane and moved by truck to an offsite location for demolition. Next, utilizing SPMTs, the existing roadway main span will then be rolled-out to the offsite staging area for demolition. Upon the removal of the existing superstructure, the substructure and bearing repairs at the piers and abutments will be completed. This work includes installing pier column extensions, abutment bearing grillages and a steel backwall extension at each abutment to accommodate the increase in vertical underclearance of the new superstructure. After completion of these tasks, the SPMTs will again be used to roll-in the proposed main roadway span superstructure from the offsite staging area a few blocks away. Next, the previously assembled end sidewalk spans will be lifted into place via crane, connected to the roadway girders and the final deck panel/floorbeam sections are to be installed. Upon completion of the erection of the full structure, the ballast will be placed and NJ Transit forces will reinstall the ties and rails before having both tracks back open for normal rail operations on Monday morning at 6am. An identical construction sequence will be used to rehabilitate the second bridge during a second weekend two-track outage a few weeks later.

CHALLENGES DURING CONSTRUCTION

Several challenges needed to be overcome during construction to ensure successful completion of the project, not the least of which is the extremely aggressive 57-hour work window to replace the existing superstructure made available by the weekend two-track outage. Further restraining this duration is the time required for the removal and reinstallation of the rail and ties by NJ Transit forces, which consists of roughly 2 hours for each track at the beginning of the outage and 4 hours for each track at the end. In order to help facilitate these efforts, the Contractor came up several time-saving measures to help remove or shorten the duration of work tasks during weekend two-track outages. This included the progressive removal of rivets at the existing pier column tops and replacement with temporary bolts before the weekend outages. Removing these rivets ahead of the time disassociated this lengthy task from the weekend operations and will significantly reduce the duration of each pier column extension replacement. Another time-saving measure was afforded by the Contractor’s request to remove the existing fascia pier columns and horizontal strut connection to the adjacent columns for rehabilitation offsite early on during construction. These pieces will then be reattached to the existing pier bents after the weekend outages,

before the fascia girders are erected. This will reduce the number of required pier column extension installations from ten to six at each bridge during the weekend two-track outages.

Another challenge during construction was the discovery of differences between the as-built plans for the backwalls at each bridge and the existing conditions in the field. Fabricated steel extensions were required at each backwall to accept the newly raised superstructure and these backwall extensions were designed and detailed based on existing as-built information for both bridges. Unfortunately, once the Contractor removed the fascia girders at each bridge, it was revealed by survey that the geometry of the backwalls was very different than what was reflected on the as-built plans. Furthermore, the horizontal and vertical alignment of the surveyed backwalls varied greatly along their length, sometimes by several inches along the front face. It became apparent to all parties that a revised detail for the backwall extensions with some geometric flexibility was required. NJ Transit, HNTB and the Contractor worked together to come up with a solution that was acceptable by design and also allowed for the Contractor's relatively easy installation during the weekend two-track outages.

SUMMARY

The Park and Watchung Avenue Bridge Replacement was a textbook Accelerated Bridge Construction project illustrating how these construction methods can be a supremely beneficial by allowing the existing bridges to remain in service until the last possible moment while the new superstructures were fabricated and erected, thereby minimizing the impacts to NJ Transit rail service and its patrons. Furthermore, the use of SPMTs to roll-out/roll-in the superstructures from an off-site staging location lessened the disruption to the city of Plainfield and the local community by reducing the duration of the road closures at the project sites and limiting impacts to local businesses in the immediate vicinity of the bridges.

G-9: ABC CONSTRUCTION SOLUTIONS

IS THAT BRIDGE OPEN YET? ABC LESSONS FROM A CONTRACTOR'S PERSPECTIVE

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ABSTRACT

Accelerated Bridge Construction using modular components has become a norm in many states. This is in no small part due to the Owner's, Engineers, Fabricators and Contractors who were willing to try new ideas and brought these visions to reality. In the 1980's and 1990's, prefabricated bridge elements and systems (PBES) were being tried, mostly in pilot projects or where rapid replacement was necessary. The replacement of superstructures in urban, highly congested areas and highways with extremely high ADT were the driving force, as multi-year staged construction projects were becoming a block to commerce and negatively impacting quality of life. The author's experience will illustrate how these early ABC projects were built from his perspective as a Contractor and a Fabricator.

INTRODUCTION

Accelerated Bridge Construction (ABC) includes several components but this paper focuses on Prefabricated Bridge Elements and Systems (PBES). Often proprietary, these systems have involved innovative people and their ideas for years. As a contractor and a precaster, the author has worked on several ABC projects and marvels at the accomplishments at those he has had the good fortune to work alongside. Starting in 1988 with the "Horseshoe Bridge" for NYSDOT to the current New Tappan Zee

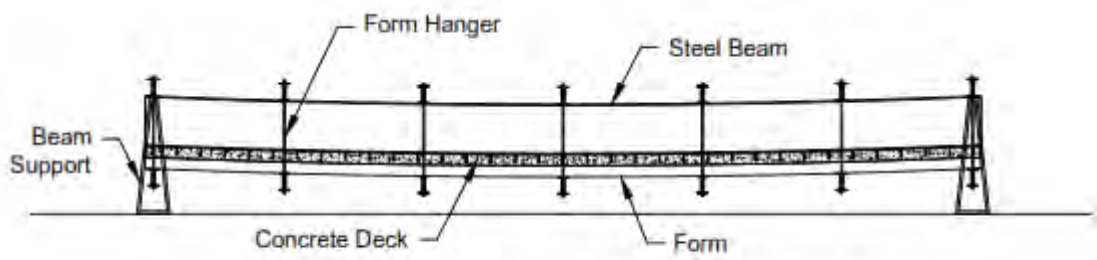
Bridge, prefabricated solutions and their challenges and advantages are presented. A common theme to all of these projects was the need for rapid bridge replacement, often with extensive liquidated damages looming. "*Is that bridge open yet*", therefore refers to the statements of all the project stakeholders. This includes the owners and engineers that have seen the need for a rapid bridge replacement, the travelling public that has been inconvenienced by the bridge's closure and the contractor whose financial loss (and sometimes gain) is directly tied to the timely reopening or completion of the bridge.

INVERSET™ AND THE ADVENT OF ABC IN THE NORTHEAST

In 1988, the Inverset Bridge System, invented and patented by Mr. Stanley Grossman, P.E. was introduced to the Northeast. Although used in Oklahoma and Texas, its introduction to the Northeast by

The Fort Miller Company of Schuylerville, NY helped solidify it as a viable option as a prefabricated bridge element. In the ten years following its New York introduction, more than 30 bridges were completed using the system in the Northeast alone, including use on the Central Artery in Boston, the Tappan Zee and Triborough Bridges.

The Inverset Bridge System uses multiple inverted steel beams (studs down) supported at or near the centerline of bearing. Deck forms are hung off the beams and the concrete deck is placed such that the weight of the deck concrete and forms prestress the steel beams. Once the concrete cures, the unit is stripped and turned right side up. In this process, the concrete in the deck is compressed such that even if supported only at midspan, the deck remains in compression. This adds to the quality, ease of handling and service life of the bridge, thus matching or exceeding the quality of cast in place concrete decks.



Inverset Bridge Module Fabrication

The first Inverset bridge was for NYSDOT in November of 1988 and set the tone for ABC construction using the modular elements. The bridge on Route 23A is a primary route to the Catskill Mountains resorts rand equired replacement before the upcoming ski season. While relatively short at 31 feet long, the bridge replacement showed just how quickly a bridge could be replaced. Since the bridge was prefabricated, it was “ready to go” when set in place with no need for special curing of the deck in the cold, November weather in the mountains of Upstate New York.

Besides the initial learning curve of building bridge components upside down, several other items became apparent that were incorporated into future projects. The steel channel diaphragms that connected the units needed to be field drilled on one end. Since the beams were in a fixed position relative to the concrete deck, they could not be moved and even a small variation in beam location would cause misaligned holes.Thus a procedure was developed where the connection plates were pre-drilled and one end of the diaphragm was also pre-drilled. Once the units were set, the diaphragm was temporarily held in place by bolts in the pre-drilled hole while the holes in the other end of the diaphragm were marked.

The diaphragm was then removed, the marked holes drilled and then installed, ensuring a perfect fit.

Similarly, since the beams are affixed to the deck, there can be small variations in the location of the bottom flange transversely as it sits on the bearing. Since most of the bearings used consisted of a steel sole plate with anchor bolts going through it and elastomeric pad, a procedure and tolerances needed to be developed. The solution was to set the anchor bolts, bearing and sole plate on the abutment or pier prior to setting the unit. The anchor bolts were set far enough apart to allow for construction tolerances and to allow welding of the bottom flange of the beam to the sole plate.

Once the concept was proven, various states in the Northeast added Inverset and similar prefabricated modules to its options for replacing bridge superstructures. But along the way, several unique situations and challenges spawned innovative details, manufacturing and construction methods to enhance the use of all types of bridge superstructure replacement systems. What follows are just a few examples.

Rockwell Falls Bridge over the Hudson River, Hadley and Lake Luzerne, NY

While Inverset had always spanned between two fixed points, the Rockwell Falls Bridge required a rapid replacement since the detour was 17 miles between the two towns and it was the only route for emergency services. When the Engineer of Record looked at a rapid replacement system, the spans were too long for Inverset to replace the existing under deck truss. Thus transverse Inverset units were developed, complete with a normal crown and sidewalks. Special steel bolsters were developed to account for the difference between the truss and the deck units and allowing for construction tolerances of the units. The bridge closure was set for 30 days with substantial liquidated damages if the contractor went over the prescribed time limit. The bridge was reopened to traffic in less than 30 days.

SR 51 over Saw Mill Run, Pittsburgh, PA

In the precast industry, if the phone doesn't ring after you've delivered the product, it usually means everything went well. But what do you do when you've delivered the product for an over the weekend closure and the day before the construction is to start you get a call that says the pieces don't fit? In the case of SR 51, that's exactly what happened. The bridge, a single span bridge carrying two lanes of traffic in each direction is on a major arterial into the city of Pittsburgh. The bridge is on a 45 degree skew and is in a superelevation transition. The plan called for removal of ½ of the bridge, cutting and rebuilding the abutments, setting the four (4) new units, and rebuilding the approach slabs between the end of rush hour on Friday night and the start of rush hour on Monday morning. In reviewing the plans prior to fabrication, it became apparent that these modules would need to be cast as "potato chips" to match the existing grade and cross-slope. By setting the beams to the final bearing elevations and making sure the form would "warp: when the deck was cast, the final shape was built in. When the contractor stated the piece wouldn't fit, he could not tell from the unit sitting on the truck that it did have the transition built in and

that when the units were set on bearings and shimmed to grade, they would follow the road profile, which they did.

Central Artery North Area, Temporary Ramps

In August of 1993, more than 30 stakeholders (including the owner, engineer, contractor and fabricators) sat around a table for a partnering meeting to kick off the construction of two ramps from a just completed tunnel section to the existing I-93 in Boston. At that time, we all knew that within a year, two ramps comprising 49 spans set on steel bents would need to be fabricated and installed. All parties committed to the project schedule and as a result everyone, working together went to work and made it happen. A total of 221 Inverset units were manufactured in NY, while the contractor concurrently cast the foundations and installed the steel bents. Production of the units required two shifts and four molds to manufacture the two units per day required. Because of all the projects completed before it, successful details, fabrication techniques (including geometry control) and erection methods ensured that the prefabricated superstructure sections would be completed on time and on budget. The two ramps remained in place until the Leonard P. Zakim Bridge was completed along with the new section of I-93 and permanent ramps replaced the temporary ramps. Some of these units have been used to build housing while the remainder sits in storage in Littleton, Massachusetts waiting to be used on another bridge.

Longbrook Avenue over Metro North, Stratford, Connecticut

When a technology becomes an accepted practice, as Inverset had become by the mid-1990's, owners, contractors or engineers ask "what if?" Their goal is to add features or cut the erection time. This project involved a single span bridge over the busy Metro-North railroad in Stratford, CT carrying commuters to and from New York City on a daily basis. In order to meet the strict closure requirements, the contract called for short closures on Saturday and Sunday mornings (between 2am and 5am). Four (4) Inverset units, 77 feet long and about 8 feet wide, were specified and fairly common by that point. However, adding a new dimension, the contractor installed all the utility brackets, conduit and piping for the various utilities at the site prior to the erection of the units. Overhang brackets and work platforms were also installed to the exterior sections as well. Catenary wires below and overhead power lines required the use of a slider beam to erect the units. In all, superstructure and utility replacement at a difficult site was made safer and quicker by the use of prefabricated modules.

Sagtikos Parkway over the Long Island Expressway

On October 6, 1994, a tanker truck collided with the barrier burst into flames on the Long Island Expressway directly below the Sagtikos Parkway. Two of the three lanes above were immediately closed on the busy Sagtikos Parkway and work began to replace the span. At 96 feet long, it was the longest

Inverset ever built at the time. In order to reduce the weight, for the first time with Inverset, a lightweight concrete deck was used, which meant adjusting all the camber formulas to account for the use of lightweight concrete in the manufacturing of the units. In addition, in order to build the bridge as quickly as possible, NYSDOT used beams from their Emergency Bridge Program. In all, working together with the contractor and NYSDOT, the bridge was replaced in about 3 months after the accident.

Tappan Zee Bridge Median Replacement

In the early 1990's, the New York State Thruway Authority was looking at both a new Tappan Zee Bridge and ways to keep the existing one in service. One of the key elements to keeping it in service was replacing the superstructure in the median along the West approach. This section was not originally designed to carry traffic, but had been retrofitted to allow for additional lanes. To develop a rapid superstructure replacement, several challenges had to be overcome. The width of the replacement was too wide for just two beams, so a three beam unit had to be developed along with a method of setting the units to ensure even bearing under dead load at all three locations. Setting the unit on the exterior two bearings, measuring the gap and installing shims to ensure even bearing, developed a procedure. The spans needed to be two-span continuous for live load, as was the rest of the structure. A unique splice detail using plates at the top and bottom of the web accomplished the splice. The existing deck overhang where it would be cut had to be supported. A complete support system was developed to support the overhangs and was grouted to ensure even bearing.

IN-DEPTH: I-95 OVER THE JAMES RIVER BRIDGE SUPERSTRUCTURE REPLACEMENT

The I-95 Bridge over the James River in Richmond, Virginia is actually two ¾ mile long, three lane structures carrying in excess of 100,000 vehicles daily. Trucks use this bridge extensively as it is the main thoroughfare between the southeast and northeast United States. The existing deck and beams were well past their service life and the substructure was also being stressed to its limits. In the late

1990's, the Virginia Department of Transportation decided that traditional methods of bridge reconstruction would take too many years and would severely restrict traffic flow causing extensive delays. They took the bold move of replacing the superstructure with prefabricated bridge elements to keep the bridge open during the peak morning and evening commutes and closing the bridge only at night. In late 1998, Archer Western Contractors was awarded the contract. The project was bid with A+B bidding ($30,000 per full nighttime closure) and contained substantial liquidated damages that went to $250,000 per night for completion more than 2 hours past the appointed opening time of 6:00 am.

Assessing The Existing Structure

In any bridge superstructure replacement, assessing both the existing condition of the deck along with ensuring the geometry are paramount to a successful project. Crews worked beneath the bridge to confirm dimensions for every beam in every span. In addition, a slow roll on I-95 was coordinated with the Virginia State Police creating a 5 minute removal of traffic at noon to allow a low altitude helicopter flyover to get a 3-D CAD Image of the bridge deck that was accurate to +/- 1/8" inch. These deck dimensions were compared with the under bridge dimensions to determine the lengths, width and orientation of all the spans.

Following that operation, the project team was able to get on the bridge during a single lane closure to assess its condition. That closure showed that the slider plate joints at several piers were completely filled with debris and "locked-up" meaning the existing superstructure could not be easily removed. We devised a plan to remove the first joint, which would allow a large enough space to remove the existing first span. Once the new first span was in place, it allowed for enough room to remove the remaining sections since the design joint was about 1".

Then, after inspecting the substructure and reviewing the plans for the existing bridge, we noticed that the steel rocker bearings were set on cotton duck fabric that had been impregnated with lead paint. Since the bearings had to be replaced, that meant preparation of a lead paint removal plan and adding time to the overnight replacement to clean the lead paint with a hazardous material team.

Developing a Construction Method

The contract documents provided for three, two girder Precast Composite Units (PCU) to comprise a typical span. The two night construction sequence suggested by the contract plans showed removing and replacing deck and stringers for one unit, then setting temporary, longitudinal grid panels between the new unit and the existing deck. The second night the contract plans called for removing the temporary grid panels and removing and replacing the remaining width of the span. During subsequent nights the joints between the decks were to be filled using a polymer concrete and then the deck post-tensioned across its width using several four strand tendons encased in a flat duct.

Shortly after award, Archer Western was able to secure land directly adjacent to Interstate 95 and immediately south of the James River Bridge. The close proximity of the casting and storage yard meant that the PCU's could be fabricated wider than shown on the plans as they would not have to be hauled over the open road and meet over-width another over the road restrictions. The adjacent casting yard

allowed for two, three girder units to comprise a typical span instead of the three two girder sections. This meant a typical section would be 22.5 feet wide, 88 feet long and weigh 140 tons.

In order to expedite the construction and to better ensure a proper fit, Archer-Western fabricated each span, as would be in place and used a match-cast joint between PCU's. Additionally, in lieu of the strand tendon, that could only be installed after all the units were set, a bar tendon was substituted. This would allow for stressing the units immediately after installation. The result of reducing the number of pieces to set, the match-cast joint and the bar tendon was a construction plan that envisioned setting one complete span per night thereby saving half of the nights required to close the bridge to traffic.

Concurrent with the decision to use the larger sections, a method for removing and installing the bridge spans had to be developed. The existing bridge posed many challenges to the method of erection. Since the bridge was on tall, slender columns with hammerhead piers, the method had to ensure that the hammerhead and column were not overloaded or would tip to one side. The existing deck was not in a condition to take the heavy construction loads (cranes, haulers, etc.), so all construction traffic was limited to driving directly over the stringers. The deck condition also limited the total weight of the crane.

Because of the time constraints, assembling and breaking down a crane each night was not feasible and limited the crane type to a lattice boom. The choices of cranes were further limited by the capacity required 75 tons at a 25 feet radius. Additionally, the crane with outriggers extended had to leave enough room for the hauler to drive by the outrigger. So, after considering many options a 150 ton lattice boom truck crane was chosen.

Once the crane was selected, an appropriate method of hauling the old pieces off and the new pieces onto the deck needed to be determined. Originally, it was decided to cut the deck into six pieces such that each beam would be removed separately with its corresponding section of deck. Therefore three beams would be removed and a new section of deck would be set in place. This meant the removal of the old deck could be done with three trailers that would be offloaded while the new deck was set in place.

The real transportation challenge was hauling the new PCU into place. The trailer had to be capable of hauling 140 ton pieces on the existing bridge. Several conventional and unconventional methods were examined with none providing the required flexibility, maneuverability and load distribution. It became readily apparent early on that a special trailer system would have to be developed. Archer-Western worked with suppliers and engineers to build a special trailer and hauling system, capable of spreading loads over a sufficient area and maneuverable enough to "parallel park" the segments within 6" of the deck opening.

PCU fabrication

Photo 1 - Aerial View of Casting Operations

Photo 2 - Typical PCU Removed From Forms

Archer Western fabricated PCU's in the casting yard adjacent to the existing bridge. Each span was setup by placing the girders in the relative positions (grade and cross-slope) as to where they would sit on the bridge. A removable forming system (required by the specifications) was used between the girders and a freestanding overhang form was used on each side.

Girders were surveyed similar to that done in the field and camber adjustments to the deck bolster were also made. After completing the survey, rebar was installed in the section of deck to be cast first. A longitudinal bulkhead was set in place at the joint between sections. The bulkhead provided the formwork for the series of female keys to be cast in the first segment. Once formwork was adjusted and rebar complete, the deck was cast using a typical bridge deck finisher.

When the concrete from the first pour reached stripping strength, the longitudinal bulkhead was stripped and a bond breaker was applied to the to the edge of deck at the matchcast joint. Rebar was then installed in the second unit and concrete poured into the second form in a similar manner to the first.

While the completed deck was curing, parapets were cast onto the decks to complete the units. Once both sides reached strength, all of the forms were stripped and the two units were split apart and moved to a storage yard. At the height of production, two complete spans were being cast per week in the two forming areas.

Once in the storage yard, the final phase of fabrication occurred. The faces of the matchcast joints were cleaned of any bond breaker. The steel bolsters were added to the bottom of the girders for adjusting the

PCU's to the proper elevation. The units were painted with the final two coats (they arrived from the fabricator with one coat of primer). Weakened plane joints were sawcut into the parapet and the parapets

were rubbed. Post-tensioning rods were installed into the PCUs on the day the segments were to be erected.

Fabrication of the northbound PCU's began in November of 1999 and continued into the summer of 2000.

Once enough units were cast, replacement of the bridge deck began.

The First Night

In June of 2000, with traffic control in place and the two pieces comprising the first span loaded onto the trailers, Archer-Western ventured out onto I-95 to replace Span 1. Unlike the typical work night, this demonstration night, free of any penalties, began Friday at 10:00pm and was scheduled for completion eleven hours later at 9:00am on Saturday. Span 1 was also somewhat unique in that it was non-composite. Several problems were encountered with the removal and replacement of Span 1. The deck was far more difficult to remove that envisioned because the non-composite deck ripped away from the stringers even though the pieces were being lifted simultaneously by the deck and the beams. Unbeknownst to all, molten zinc had been previously poured and solidified into the hole in the masonry plate around the anchor bolts in the holes of the existing bearings, making the bearings extremely difficult and time consuming to remove. Sawcutting the deck into six individual pieces took too much time to cut and to remove, plus the pieces were unstable. All of these items created a scenario that had the first unit installed after the anticipated completion of replacing the entire span. The condition of the deck and its non-composite nature forced the replacement of the other half of the span without returning the road to traffic. By the time the second half of the bridge was replaced, the bearings all welded and the bridge opened to traffic, it was 12:30am on Sunday. The replacement had taken 26-1/2 hours.

A Reformulated Plan

With the first night's work complete, it was obvious to all involved that some changes had to be made to the work methods and scheduling. The contract allowed for a second demonstration night if the contractor's scheme was not successful on the first try.

It was agreed by all that the next span would be done in halves, with the temporary grid as proposed in the original contract plans installed. Also, since there were three girders to remove in each half, the exterior girder would be removed with the parapet, then the second and third would be removed together along with approximately 16 feet of deck. Coupled with the fact that Span 3 was composite, it was felt that this method would speed up the removal and provide a more stable piece.

With the new plan in place and another chance to try, Archer Western's crews returned to the bridge on a subsequent weekend and successfully removed and installed span 3 on two consecutive nights. The work was completed in a time frame that suggested that each span could be replaced in the allotted time.

The next five spans were replaced in two evenings per span, with significant improvements each night.

By the time the eighth span was to be replaced, the original idea of replacing an entire span in one night was tried again. This time, the span was completely replaced in one night, including post-tensioning and removal of MOT. This night set the stage for the rest of project. As time rolled on and crews became more familiar with the equipment and the work, an entire span was replaced in approximately 7 to 8 hours, including all traffic control.

Photo 3 - Setting Span 3 in Place

The Real Lesson Of Time Critical Bridge Replacement

While it is the desire of every owner to replace a critical structure as quickly as possible, minimizing the inconvenience to the motoring public, it is imperative that all stakeholders work together to make the project a success. A "gotcha" mentality of any stakeholder ensures that no one will win. Since VDOT and the Engineer of Record had the foresight to include two, no-penalty demonstration periods, it removed the much of the animosity that would have been prevalent between the two sides based on all the issues encountered on the first night. In order to better understand the process and some of the nonobvious risks involved consider the following thoughts on overnight bridge replacement.

Since Archer-Western removed a section of bridge every night, once a sawblade was put into a deck, there was no return. Impending thunderstorms, equipment failures or traffic accidents could not be used as reasons for time extensions. Nor could the traffic control simply be picked up and allow traffic back on the bridge. There is no “we’ll get that tomorrow” on this type of construction. No matter how many scenarios that are contemplated, there will always be something else that will cause a problem, a bit of

2:00 a.m. engineering. As contractors have become accustomed to this type of construction, they have become better at anticipating the unexpected.

The very nature of this time critical work makes it difficult for the inclusion of subcontractors.

Subcontractors do not have and will not accept the same level of risk associated with this type construction and therefore will tend not to have the same urgency that the general contractor will. Archer-Western personnel, including the crane operators performed all operations except hauling the existing deck from the bridge and the lead removal. Most subcontractors were very hesitant to even look at work of this nature, cognizant of the fact that they may be responsible for all or part of the penalties in a given night and were not comfortable with that amount of risk.

Finally, the cost of mobilizing and returning an army of vehicles each night is a tremendous undertaking.

Each night, a barrier moving machine, three cranes, four flat bed trucks each with trailers, two forklifts, two tractor-trailers, light plants and countless pickups entered onto the bridge during a five-minute traffic stoppage. As you can well imagine, this parade was an impressive sight and required a great deal of coordination.

In all, this project was a success and the Virginia Department of Transportation took the lessons learned and recently replaced 10 smaller bridges on I-95 in downtown Richmond.

WHAT ABOUT THE JOINTS?

Anytime modular components are used in construction, there is always concern about the joints.

Additionally, many modular bridge systems have longitudinal joints that come all the way to the riding surface. While several different approaches have been taken to joints, there are a few that have worked well over the years. The common factors in all successful joints are simple details with easy to use materials.

Longitudinal joints are usually not in the wheel path, although it must be considered. Therefore proper location of these joints is a key to a good joint. Inverset and other similar composite superstructure elements usually have a short overhang from the supporting member (steel beam) to the joint. Testing and real world experience shows that the diaphragms are very effective in transferring loads between the beams of adjacent units, thus minimizing the differential deflection between the two edges of the concrete deck, even without a grouted keyway. As a result, using a simple grouted keyway along with a flexible joint seal at the top appears to provide good results. The photo below taken in April of 2015 shows a joint of an Inverset bridge installed in 1995. While the joint isn't perfect, it is still serviceable and has endured 20 harsh winters of the Adirondack region of New York.

Photo 4 - Crescent Ave Over I-87

Photo 5 - Close-Up of Joint

What if the existing bridge geometry doesn't allow for a short overhang. The I-95 James River Bridge in

Richmond did not allow for short overhangs. The engineer of record proposed the use of a polymer concrete post-tensioned joint using multi-strand tendons. As noted, this was changed to a match-cast joint with shear keys and used high strength bars to post-tension the joint opposed to strands. This change did not change the design, since the high strength bar had the same capacity as the four strand tendon but greatly accelerated and simplified the construction. Today, 15 years later, the joints on the bridge are holding up well and VDOT recently used the same type of joint for the replacement of 10 other bridges on I-95 in Richmond.

WHEN ABC IS SOP

The acceptance of prefabricated bridge elements for highways has steadily grown over the last 30 years.

Increased traffic and greater expectations from the travelling public have helped propel this trend.

Originally, toll authorities looked at these options as any additional cost from this type of construction was offset by the loss in toll revenue. However, railroads without the luxury of detours and extreme costs when freight trains aren't running long ago adopted ABC concepts. To the railroads, a 10hour bridge replacement is the norm. In October of 2014, the author witnessed the replacement of a Canadian Pacific Railroad Bridge in Fernie, B.C. with a prefabricated Hillman Composite Beam Bridge.

When we arrived in the morning at the staging yard in Fernie all the necessary equipment was lined up and ready to go. The two rail mounted cranes, one to remove the existing bridge and one to install the new bridge, the new bridge modules, each on their own flat car, and all the other equipment and material needed to replace the bridge. Shortly before the last train went through and the work could begin, a meeting with all personnel was held in the yard to discuss the objective for the day (replace the bridge within the 10-hour track blockage) along with the hazards of the operation and ways to stay safe. Once the final train went through around 8 am, dispatch turned the track over to the crew to begin work. As the trains rolled out, all in sequence, the dispatcher asked the crew leader, how much longer will the bridge be closed? This was the first of several calls from dispatch wanting to open the bridge as quickly as possible.

When the modules arrived on site, the old bridge was already removed. The rails had been cut and removed with the ties along with the removal of the two steel plate girders. The structure crew was already at work drilling holes for anchor bolts and grinding out the old pedestals where the steel beams had sat for 100 years. By lunchtime, the abutments were ground, the new anchor bolts were installed and the crew was waiting for some grout used to level out the abutment to set. Simultaneously in the morning, another crew rebuilt the track section by taking the rails off the old ties and installing them on new ties. Shortly after lunch, the two units were set in place in about 1 hour. Each module consisted of one 6'-6" wide HCB with a 7'-0" wide concrete deck and a 2'-2" high ballast curb. The 33'-0" long pieces weighed approximately 22 tons.

With the modules in place, the next step was to install the keeper angles on the abutments, and cover the joints with galvanized steel plates. The track section was then set in place on the bridge and the rails spliced at either end of the bridge where they had been cut earlier that morning. Ballast was then placed on the bridge. Immediately following that, a track profiler was used to compact the ballast and profile the track. In all, by about 5 pm, the bridge was reopened to traffic.

This operation, with its various crews and equipment, was like watching an orchestra. Each of the participants were experienced with the job they were doing, all were moving together to make the bridge replacement happen, like they had done it several times before (which they had). It showed that

Accelerated Bridge Construction can be Standard Operating Procedure.

Photo 6 - Canadian Pacific Bridge Replacement - All in a Day's Work

CONCLUSION

Over the past 30 years, innovative contractors, engineers and owners have created a record of successful ABC projects using precast bridge elements. Learning from these projects, the good and the bad, means that costs will become less and the value will become even greater. These elements, the details used and the erection techniques are becoming more commonplace. Highway bridge replacement isn't quite where railroad bridge replacement is today, but it is gaining on it and perhaps someday, ABC will be SOP. The lessons learned and increasing adoption of these products and techniques show that it likely will. The answer to the question "Is that bridge open yet?" will be "YES".

G-10: PBES CONNECTION DETAILS AND MATERIALS

RAPID BRIDGE DECK JOINT REPAIR AND REHABILITATION

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ABSTRACT

Bridge deck joints serve an important function for bridge performance by accommodating bridge movement due to thermal expansion & contraction and movement from dynamic loads. In Iowa, the joints typically serve a secondary purpose of preventing the passage of water often containing deicing chemicals and other corrosive chemicals that can cause severe deterioration to the underlying bridge components. Bridge deck joints are a component of the bridge that take a considerable amount of abuse from impact loading by trucks and snow plow blade strikes that cause damaged and broken joints.

Additionally, the joints are located in a severe exposure location and even the best joints need repair and rehabilitation over the life-cycle of the bridge.

On the Iowa primary road system nearly 400 bridges contain legacy steel sliding plate joints, many badly

in need of replacement. Joint repair and rehabilitation has become a common project for the Iowa Department of Transportation (DOT) in order to maintain the transportation system. Many of these joint repair and rehabilitation projects are located on high volume interstate roadways and in urban areas where staged construction or a detour has a highly detrimental effect for maintaining traffic mobility for the Iowa DOT's transportation customers.

Research was commissioned to develop a rapid, effective, and economical approach to repairing and replacing bridge deck joints. A baseline for an accelerated bridge deck joint replacement was established by observing various joint repair and rehabilitation construction projects including a staged weekend replacement project on I-380 in Cedar Rapids, Iowa on a stretch of interstate roadway that accommodates 74,000 vehicles per day. Workshops with subject matter experts from design, construction, maintenance, and materials were held to identify approaches for improvement in repairing and replacing bridge deck joints. Suggested approaches that resulted from the workshops and other investigations are:

- Eliminate the requirement to maintain existing rebar and make greater use of the practice of drilled and bonded dowel bars.

- Remove the smallest amount of concrete possible. Cut off embedded anchorages and leave them in place if they are not in the way.

- Redesign strip seal anchorages to allow for smaller demolition areas.

- Investigate hydrodemolition, because it may provide a considerable decrease in removal times but also has a number of drawbacks.

- Consider the use of polymer concretes for joint headers that cure in as little as 4 hours. Plan for the use of high early strength concrete because it is effective in some cases, but not always readily available without prior planning.

This paper describes in greater detail the background, research methodology, findings, and conclusions of this investigation.

INTRODUCTION

Bridge deck expansion joints are the components of a bridge that allow for movement of the bridge deck due to thermal expansion, dynamic loading, and several other factors. More recently, expansion joints have had a secondary function of preventing the passage of water. This water often contains deicing salts and other corrosive chemicals that are harmful to the substructure of the bridge.

Expansion joints are often one of the first components of a bridge to fail. Failure can be due to increased traffic loading, component fatigue, low quality work, or several other factors. Joint failure can lead to increased damage to bridge substructures including rust formation on metal bearings as well as increased spalling on precast beam ends, concrete abutments, and concrete piers. To prevent further bridge damage, joints are often repaired or replaced.

Joint replacements are particularly problematic construction projects, often requiring traffic closures to allow work completion. Traffic closures are undesirable and often require staged jobs and difficult working conditions. Completing work during low traffic periods, nights, and weekends can help alleviate traffic concerns. However, it is challenging to complete a repair in a very short period of time or at night while still maintaining the necessary joint quality. Improved methods to rapidly repair and replace bridge deck expansion joints are desirable.

A research project was commissioned to establish a baseline for accelerated bridge deck joint replacement by observing a bridge deck joint replacement on NB I-380 in Cedar Rapids, Iowa an interstate roadway that accommodates 74,000 vehicles per day. Following the establishment of a time baseline for an expansion joint two workshops with subject matter experts from design, construction, maintenance, and materials were held to identify approaches for improvement in repairing and replacing bridge deck joints.

BASELINE FOR ACCELERATED BRIDGE DECK JOINT REPLACEMENT

In construction, challenges exist in communication and understanding between the design engineers and the workers completing the physical repairs in the field. Design changes can help expedite field work, but existing processes to replace expansion joints must be understood before changes can be made.

Conversely, many jobsite supervisors may also have ideas that can facilitate more rapid completion of the repairs, but lack the engineering knowledge required to ensure that a design meets required standards for safety and durability. Thus, an objective of this observation of an accelerated bridge deck joint replacement is to make engineers more intimate with the specific means and methods currently used during rapid joint replacement projects.

The Northbound I-380 joint replacement research targeted activities that occurred during the second year of a two-year project involving the complete removal and replacement of several expansion joints along I380 through Cedar Rapids, Iowa. The joint replacement specifically observed, designated as Joint A, was immediately before Exit 19A on Northbound I-380.

In total, five expansion joints were to be replaced as part of this project over three consecutive weekends.

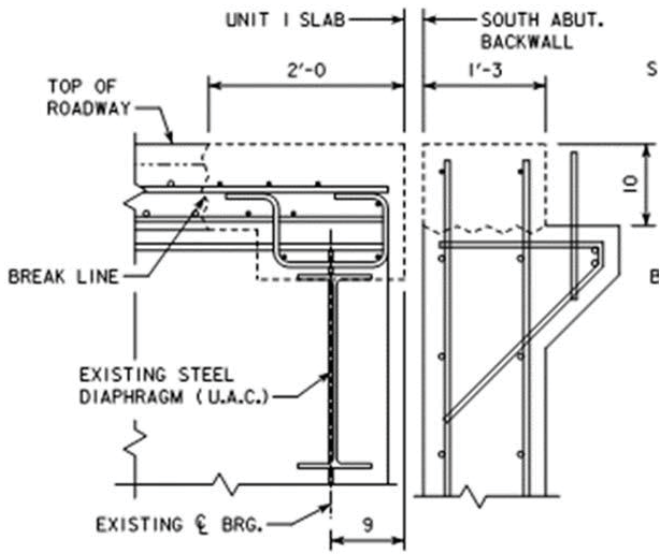
Observations of the project were made during the first weekend. Detailed records were only kept for Joint A. However, some comparisons were also made throughout the project involving Joint D.

The initial staging during the first weekend of work entailed replacing both joints on the exit ramp, Joints D and E, as well as half of Joint A. The remaining half of Joint A, as well as Joint B and C, were replaced in sections over the next two weekends. With this staging plan, only the exit ramp would be entirely closed to traffic only for a single weekend. This closure could not be avoided due to the width of the ramp. For the remaining two weekends, at least one lane would always remain open.

Joint A was an old sliding plate joint still in use long past its service life. Overall, the joint did not appear to be in extremely bad condition, because only an approximately one-foot long section of plate had broken loose. However, when the top steel plate was removed, it revealed a considerable amount of rust

that had built up. There was enough rust between the plates of the expansion joint that both the Iowa DOT inspector and the author doubted that the joint had been properly functioning in years. Other joints in this project, and Joint E in particular, exhibited much more severe failures that ultimately prompted the replacement.

The old sliding plate joint was to be replaced with a new strip seal expansion joint. Concrete removal would consist of the top of the backwall from the existing riding surface to the top of the paving notch and the end two feet of roadway concrete (see Figure 1).



Schematic: Iowa DOT

Figure 1 - Typical I-380 Concrete Removal Cross Section

Unlike other joint replacements, this job did not require the removal or replacement of the approach slabs, paving notch, or the entirety of the backwall. Embedded rebar was to remain for the reconstruction of the joint. Any bars not embedded in the concrete were to be removed and replaced with epoxy-coated bars, which largely included the existing hoops and longitudinal bars. The new expansion joint and reinforcing steel (rebar) would be formed and constructed using a high early strength concrete mix. Previous tests on the concrete mix had resulted in the development of a maturity curve that indicated the required compressive strength of the concrete of 4,000 psi to be reached in 9 to 12 hours.

One of the main focuses for observing the replacement of Joint A was to find the length of time required to complete specific construction tasks. Knowing the typical length of a construction task greatly facilitates efforts to reduce the overall time of a joint replacement project. The longer the task, the more

potential that task has for reducing the overall time of the project. If the task only takes a few hours, reducing that time is unlikely to shorten the entire project considerably.

Traffic closures were allowed from 7:00 p.m. Friday evening until 6:00 a.m. Monday morning. Thus, traffic control measures started precisely at 7:00 p.m. Friday evening. Traffic control initially consisted of signage that directed traffic to change lanes, as well as traffic cones to designate closed lanes. The initial use of traffic cones allowed equipment mobilization to proceed as soon as possible after the 7:00 p.m. project start time.

On such projects where a considerable amount of work is done in a short amount of time, it is best to complete tasks concurrently with other tasks as often as possible. After traffic had been completely redirected out of the work zone, traffic cones were replaced with jersey barriers to increase the safety of the jobsite. As seen in Table 1 traffic control took about four hours of the project time to complete.

However, since traffic control worked concurrently with equipment mobilization and demolition, it had little impact on the overall project time. Thus, the overall project time would not be reduced by reducing the time to install traffic control.

Table 1. Total construction task lengths

Activity	Total Hours
Traffic Control	4
Equipment Mobilization	2
Hydrodemolition of Joint A	14
Demolition with 15lb Chipping Hammers	6
Total Demolition Time	20
Formwork and Rebar Placement	13
Concrete Placement and Cure Time	11

Equipment mobilization began shortly after traffic was completely rerouted, which was about an hour into the project. This job was unique in that the contractor utilized hydrodemolition for the majority of the concrete removal on Joint A. The contractor utilized an Aqua Cutter from Aquajet Systems AB.

This system requires not only the aqua cutter but also a water storage truck and several trailer-mounted pumps to provide the necessary water pressure. The contractor also mobilized several towable air compressors and several 15-lb chipping hammers.

The aqua jet equipment took several hours to set up and properly align with the limits of demolition before the contractor could begin cutting. While this happened, the steel plates that formed the existing expansion joint were removed with an oxy-acetylene torch. It was explained by the supervisor that the aqua cutter would not be able to remove any concrete below the steel. Thus, the more concrete that could be exposed, the less concrete would need to be removed by hand.

A moveable cage, which was essentially a few aluminum fence posts with several layers of orange snow fence, was placed around the aqua cutter on three sides. It was explained that during demolition, small particles or broken concrete may be thrown into the air. The particles would be small, ejected with little force, and of no danger to the workers or observers. However, these small particles could potentially cause superficial damage to passing traffic and damage to passing traffic needed to be prevented.

Demolition with the aqua cutter started promptly at 10:30 p.m., but was stopped after a short time. It was discovered that, upon removing the bottom layer of concrete, the water jet was digging a trench in the ground beneath the bridge. This had been anticipated by the contractor as a potential problem and the delay was short while sections of scrap steel plate were placed beneath the sections that were to be removed. The demolition process then continued. The aqua cutter had a demolition width of about 5 ft. After completing the removal between the required limits, the machine was moved to the side, realigned with the previous sections of demolition, and restarted. Hydrodemolition of Joint A took place for about 14 hours (see Figure 2).

Image: Adam Miller, ISU
Figure 2 - Joint A After Hydrodemolition

The aqua cutter was capable of removing most, but not all, of the concrete necessary to replace the joint.

In particular, the aqua cutter could not remove the concrete within about 8 in. of the curb, as well as the curb itself. While not of concern to this project, this area near the curb may be larger if the joint is at a skew to the curb. There was also a small section of concrete beneath the existing joint that could not be removed with the aqua cutter.

The remaining concrete was removed with 15-lb chipping hammers. This was much slower than the removal by hydrodemolition, but also consisted of concrete often in confined areas and corners. Removal with chipping hammers was about a 6-hour task, bringing the total time for demolition to 20 hours.

Removal of Joint D had been done with 15-lb chipping hammers until the water jet had finished on Joint A. At this point, about a third of Joint D had been removed with 15-lb chipping hammers. The water jet was then moved to Joint D to finish removal of that section, while the 15-lb chipping hammers were moved to Joint A to remove the remaining concrete.

The formwork installation started when about half of the existing joint had been entirely removed.

Formwork was not complicated for this project and consisted of plywood supported by 2x4 lumber. Some of the sections had been precut and preassembled to expedite the process of installing the formwork. The concrete profile was identical to the section to be removed although the reinforcing steel (rebar) layout had changed slightly for the new joint. This rectangular layout was ideal, as it avoided the need to build formwork with any angles other than 90°. Other shapes, such as the angled profile of many paving notches, are more time consuming to construct than simple rectangular sections. Formwork was all placed by hand as the sections were not large enough to require any additional equipment.

The installation of the new reinforcing bar proceeded shortly after the bottom sections of formwork had been placed and supported. Waiting until the forms are in place allows the reinforcing steel to be supported by the forms at the proper elevation, by the use of rebar chairs, and ensures that proper cover requirements are met the first time the rebar is installed.

On this particular job, the contractor had to install, then remove and reinstall the rebar several times before the layout was correct. Overall, the additional effort involved in installing the rebar probably added several hours to the project length. The Iowa DOT inspector commented that the workers appeared inexperienced with rebar placement.

The reinforcing steel (rebar) was placed and tied together by hand with epoxy-coated rebar tie wire. The expansion joint extrusion was set in place with the reinforcing bar. The joint extrusions were separated by a piece of three-quarter inch foam insulation and then clamped together. The foam insulation would

maintain the proper spacing while the concrete was poured and was both compressive and easily removed in pieces if the deck was to undergo expansion before the insulation was removed.

While the reinforcing steel was being placed, the end sections of formwork and bulkheads, again constructed out of plywood and dimensioned lumber, were installed. Formwork and rebar installation finished in the early hours of the morning and no additional work was completed on Joint A until later in the morning when the concrete batch plant opened to provide concrete. At this point, Joint A would easily be finished before the set deadline as long as the concrete was delivered to the site at a reasonable time.

Concrete placing and finishing was an easy task on this project. Concrete arrived at the site promptly at 10:00 a.m. A high-range water-reducing admixture, as well as other chemicals, were added to the concrete on site immediately before the concrete was placed. The engineer that designed the concrete mix stated that the concrete would begin to set initially about 25 minutes after the chemicals were added, with previous tests showing required strengths being achieved in about nine hours.

The concrete pour was much more organized than the rest of the project and the construction laborers appeared to be very experienced with concrete pours. Immediately before concrete was placed, a thin layer of grout was placed by hand on all existing concrete faces that would adjoin the new concrete.

Concrete was then placed directly from the truck into the formwork and vibrated with a flexible shaft vibratory compactor. Once the concrete was placed and vibrated, the clamps holding the joint extrusions in place were removed. Even though the concrete had not yet set, pressure of the concrete behind the extrusion would hold the joint against the insulation separator.

The concrete was then finished by hand, first with wooden floats and then with magnesium finishing trowels, to provide a nice smooth riding surface. Curing compound was sprayed on the surface of the concrete, and the joint was left to cure.

Some conclusions were made during the observations of this jobsite. The conclusions were reached during downtimes during discussions between the research team, the Iowa DOT inspector, and the jobsite supervisor.

- Demolition was the single longest construction task with concrete cure time taking the second most amount of time.

- There was no clearly obvious way to precast an expansion joint.

- General formwork shapes could be prebuilt, but complete prebuilding of formwork would be difficult.

- The prebuilding of formwork was a particularly prevalent topic. The same contractor had completed an identical job on the southbound lanes of I-380 the summer before and had not premanufactured any formwork. To save time during the observed job, general formwork shapes had been pre-constructed before the job began.

The discussion focused on the possible use of a pre-manufactured steel form that could be erected much more quickly. However, this idea was discarded as nearly impossible because, even though the Iowa

DOT provides standard profiles for bridge members, the final dimensions often vary slightly. It would take a substantial number of different forms to have a form that would work for almost every bridge. Thus, it was just easier, cheaper, and seemingly slower to use plywood formwork and construct a portion of it during the job.

RAPID REPLACMENT OF EXPANSION JOINTS WORKSHOPS

Two workshops were held with subject matter experts from design, construction, maintenance, and materials were held to identify approaches for improvement in repairing and replacing bridge deck expansion joints. The first workshop was held December 4th, 2013 with the primary purpose of creating a collaborative environment where subject matter experts from design, construction, maintenance, and materials could discuss bridge deck expansion joints and the problems they encounter in multidisciplinary teams. The primary focus was on expediting the construction process but any and all ideas to improve expansion joints in general were considered. The top results of the first workshop are summarized as follows:

Assess Existing Joint Behavior

Existing expansion joints are selected largely based on the expansion distance required by a bridge.

Tests should be conducted to measure the actual joint expansion distance, which can be compared to the theoretical joint expansion distance. This should be particularly noted with respect to the age of a bridge.

The pertinent question to ask is if the required expansion for the bridge reduces with age so that, when joints need to be replaced, they may be replaced with a more easily maintained joint that allows less movement (e.g., a finger joint to a strip seal or a strip seal to a semi-integral abutment).

Develop Standard Details for Precast Joint, Paving Notch, and Approach

Discussions with the contractors present during the workshop revealed that concrete cure time may consume as much if not more time than concrete removal and that this time spent waiting for concrete to cure could be reduced with the use of precast members. Time savings could be increased by an even larger amount if existing rebar in the concrete that is to be removed is not required to be maintained as protruding rebar to facilitate lap splicing.

Increase Use of Semi-Integral Abutments

During discussions with the contractor representatives who were present, it was stated that integral abutment and semi-integral abutment expansion joints are the favored joints among contractors. These are the easiest and quickest joints to erect, as the sections are more or less just rectangles with a paving notch. The standardization of details makes these joints faster and easier to construct than stub-type abutments.

Eliminate Strip Seal Upturn at Gutter and Develop Drainage System

The main cause of deterioration in strip seal expansion joints is from the accumulation of debris in the neoprene seal. Currently in Iowa, strip seal joints are designed to prevent the flow of water through the joint. Sand, deicing salts, and other debris are collected in the joint during the winter months. This debris can cause splits under traffic loading, or prevent full expansion during summer months.

The workshop participants concluded an alternate system should be developed that reroutes water through a drainage system depositing contaminated water away from important structural members while still maintaining a watertight membrane at the joint. The flow of water would also prevent debris accumulation by flushing debris from the joints.

Develop a Proactive Maintenance Program

Currently joint components are replaced when they fail. Most commonly this means failure of neoprene glands in strip seals and compression seal glands. The wait time between the discovery of these failures, programming, bidding, and finally replacement can mean a significant amount of time that a joint is functioning but not watertight.

It was suggested that most strip seal glands fail at about 15 to 20 years while compression seal glands fail most commonly after 10 years. Thus, glands should be replaced proactively to prevent failure instead of waiting until failure has already occurred.

Evaluate the Use of Dowel Bars and Fast-Curing Concrete

Demolition and cure times are two of the longest tasks during a joint replacement. In general, the Iowa DOT prefers to maintain the existing vertical bars in stub abutments to allow the usage of lap splices. This requires that the concrete be removed from the bars while the bars remain in largely good condition.

Smaller demolition tools must then be utilized to remove the concrete, slowing the overall pace of the job. Allowing the complete removal of the vertical bars will allow removal times to be significantly shorter, or about one work day in the researcher's prior experience, instead of the several days that it now currently takes. The use of new reinforcing steel doweled and grouted into the old footing will add additional rebar placement time, but this added time should be a fraction of the time saved in removal.

Develop a Mechanical Attachment for Future Joint Replacements

Current expansion joints are generally cast integrally with the concrete bridge deck. When joint replacements are necessary, this requires that concrete be demolished, new rebar placed, formwork erected, and new concrete placed. A retrofitted mechanical attachment would alleviate future problems.

Similar to replaceable parts in a mechanical system (car, machinery, etc.), these joints would be designed to be easily replaced.

Evaluate Concrete Mixes and Better Specify Proper Use of High Early-Strength Concrete

During the workshop discussions, the contractor representatives noted that concrete cure time may extend joint replacement schedules almost as much as concrete removal time. Concrete mixes that reach usable concrete strengths in as little as 24 hours or less are currently available. However, in rural areas where such mixes are not used as regularly, necessary maturity data may not be available. In addition, guidelines should be developed to better specify when certain mixes are truly beneficial. An 8-hour concrete mix would be beneficial on an overnight project, but would have no benefit over a 24-hour concrete mix on an extended closure.

Determine Allowed Movement for Different Concrete Mixes

Concrete mixes of cement, aggregate, and polymer have been used previously to serve the same functions as an expansion joint gland to accommodate small movements in bridge decks. Other mixes of asphalt binder and aggregates are available that have been proven useful in accommodating expansion.

However, the Iowa DOT has not done an in-depth study for the amount of expansion that could be allowed for the many different concrete mixes that currently exist. Some polymer and asphalt concretes may allow sufficient elastic movement for short bridges to allow the elimination of expansion joints

altogether. In particular, the Michigan DOT has been experimenting with the use of engineered cementitious composite link slabs in bridge deck rehab as opposed to traditional expansion joints.

Develop Emergency Procedures for Evaluating Necessary Quality of Repair

During prior research tasks, trips, and conversations with maintenance forces, it was discussed that, at a handful of times during the year, emergency repairs were necessary on expansion joints. The repairs often included the removal of significant portions of the existing back wall and large sections, if not the entirety, of the expansion joint. Repairs were often completed in as little as eight hours to restore use of the bridge to traffic.

Both the Iowa DOT maintenance personnel and the contractors at the workshop noted that the concrete removal during these repairs already constituted about half of the work required in a normal joint replacement. Both also seemed to agree that, with a longer closure time (possibly as little as 2 to 3 days) and a few other changes, these temporary repair efforts could easily be expanded into complete joint replacement projects.

A follow up workshop was held February 18th, 2015 to focus specifically on the possibility of developing a standard detail for a precast or mechanically attached replacement joint. A smaller group of subject matter experts was gathered primarily with experts from the design and construction disciplines to accomplish the objective. The workshop participants were separated into two groups to independently develop a detail for rapidly replacing the expansion joints. For time reference, the length of the joint replacement on I-380 was utilized as a baseline of about 50 hours, which can be accomplished in a single weekend. For a proposed detail to be worthwhile it would either have to have a shorter schedule than this baseline, or provide other substantial benefits such as decreased cost or increased longevity. The two group concepts developed are as follows:

Group A Joint Concept

The concept in group A began by discussing what expansion distance was actually required for the typical joint replacement in Iowa. An analysis of Iowa DOT bridge data found that of the 379 steel girder bridges in Iowa with existing sliding plate joints, 193 bridges were less than the 300 ft maximum length for an integral abutment on a 45 degree skewed bridge. For a maximum bridge length of 400 ft, which must have a zero degree skew in order to be an integral abutment, a total of 299 of the 379 bridges met this requirement. These length and skew requirements are available in the Iowa DOT LRFD Bridge Design

Manual, Iowa DOT (1). This larger number includes some bridges that may have still been disqualified from using an integral abutment due to skew and length requirements as well as end span requirements.

This information is provided to reinforce the fact that many of these bridges, if constructed today, would be designed with integral abutments. However, the existing foundations do not allow for integral abutments, and the time required to convert these bridges to semi-integral abutments makes this option unappealing if time is a constraint. A joint is required, but these bridges are far from requiring the full 5+ inches of expansion distance that strip seals are capable of providing. So, what if the joint at the abutment was eliminated entirely?

A working idea based on the question asked at the end of the previous paragraph eventually evolved into the concept shown in Figure 3 below. Since the initial thought behind the workshop involved precast components, it was thought that a new precast approach slab could be utilized to span the existing abutment backwall and that the expansion joint could be pushed out past the bridge substructure onto the approach panel where leakage is less of a concern for deterioration. A precast panel approach would also accommodate staged jobs where traffic must be maintained. The precast panels could be constructed in approximate lane widths allowing one lane to be replaced at a time. By spanning over the backwall the design would also eliminate the worry of finding a failed paving notch during the joint replacement process. In fact, it would entirely eliminate the need for a paving notch to support the approach slab as the approach slab will now bear directly on the bridge girders. There were a few immediately noted disadvantages involving this detail. First, one end of the precast approach slab would bear on the bridge substructure. The other end would require a sleeper slab which would have to be constructed to support the dead load of the slab and live load of the traffic. The second concern was that the end of the bridge may need to be strengthened in order to support the additional live load of traffic across the additional span. However, it was predicted that any necessary strengthening could be accomplished beneath the bridge without disrupting traffic. The last immediate concern was that if the sleeper slab were to settle, the slab that was simply supported above the existing backwall could now come into contact with the backwall. This could eventually result in the backwall or slab failing from an additional applied load that they were not designed to carry.

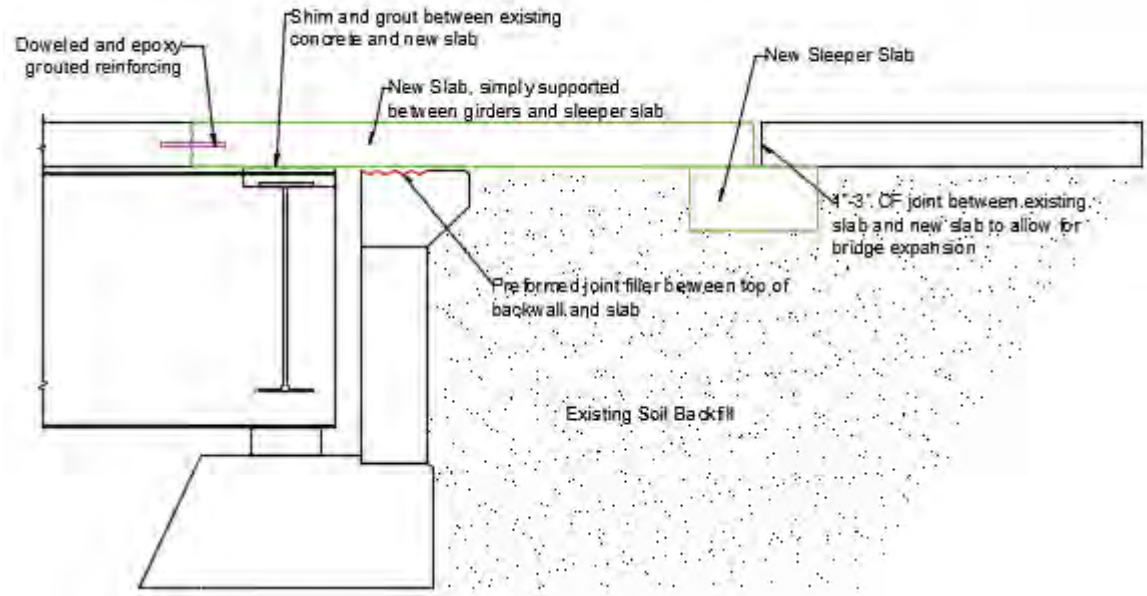


Figure 3 - Tied Precast Approach Slab Concept

When all of the workshop participants were together the feasibility of this idea was discussed in length.

The addition of a new approach slab would require the removal of the existing slab. It was thought that this extra amount of demolition would not add a considerable amount of time to the project duration. In fact, on a previous rapid construction project to quickly remove the existing approach slab, it had been sawcut into manageable sections during the evening prior to the project when traffic was light. Traffic was allowed to travel over these approach slab sections for the next day causing no disruption to traffic. When the project finally began, an excavator with a claw attachment was utilized to pick up individual sawcut approach sections and the approach slab removal was completed with little hindrance to the project schedule.

The connection between the existing deck and new approach slab could be difficult to construct.

However, if a precast slab is used, the existing longitudinal bars in the deck will have to be removed. It would be difficult to accurately determine where the existing bars are located to correctly construct the precast slab. Thus, the dowel bars will have to be located in the precast slab and be grouted in the existing deck to achieve the required development lengths and connection between the approach slab and the deck. The workshop participants realized that there was a new problem of aligning the bars properly without damage. The bars would have to be located approximately at the mid depth of the span to avoid spalling of the bridge deck. If holes were drilled horizontally into the bridge deck, the approach slab would have to

be tilted or slid into place to insert the dowel bars into the holes to be grouted. Grout tubes would then also be required to fill the holes and develop the proper connection strength. Getting the slab lined up this way would be incredibly difficult. Sliding the slab into place risks damaging the reinforcing bars, slab, and bridge substructure, while lifting the slab into place requires the slab to be picked up and placed at an angle which would be difficult.

Instead of drilling holes, slotted dowels could be used. Slots could be cut into the bridge deck, the slab and dowel bars could be set in place horizontally, then these slots are filled with high strength grout to develop the connection between the new reinforcing and existing deck. However, cutting these slots may require cutting one or more of the transverse reinforcing bars as seen below in Figure 4. This detail will require further investigation.

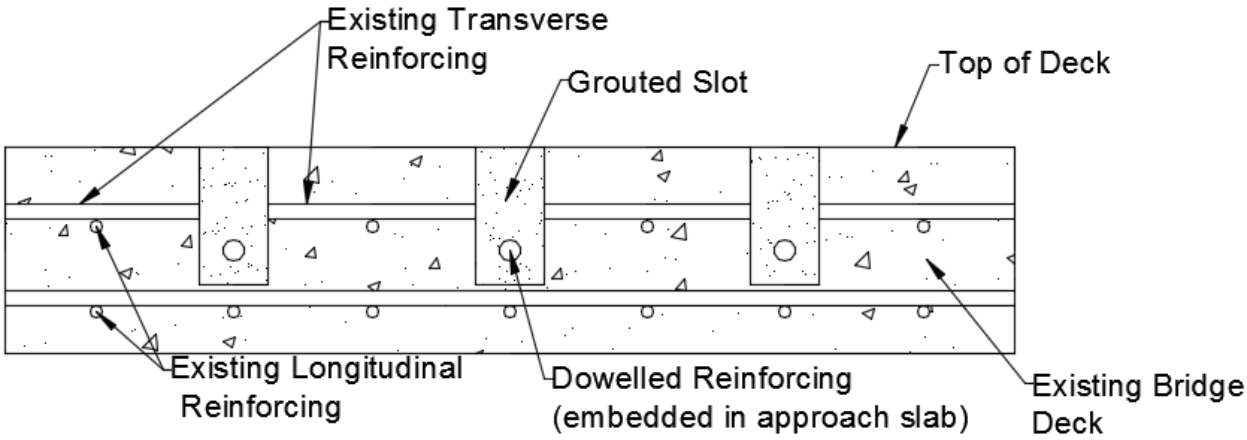


Figure 4 - Slotted Dowels

There were other potential issues with this joint. Placing two slabs together in this fashion will create a cold joint in the deck. The point of this new slab is to move the potential leaks away from the bridge deck where any future leaks would not be a serious concern. Further investigation will be required to determine if this cold joint will remain closed tightly enough to prevent leakage at this cold joint. If the cold joint will not be watertight then instead of moving the potential problem away from the bridge, the problem would have only been moved to a different location on the bridge.

Group B Joint Concept

The concept from the second group can be best described as less is more. Since concrete demolition is the most time consuming activity, less concrete removal means less time. From prior experience in

Kansas, a contractor had constructed many joints that only required removing a 4 - 5 inch depth of bridge deck as opposed to a full depth deck removal. Any reinforcing or anchorages that were encountered during concrete removal were simply cut out and removed. Existing joint anchorages take a great deal of work to remove. Sliding plate joint anchorages in particular, are bolted to the top flange of the girder and require the removal of a considerable amount of concrete. The size and depth of this anchorage can be seen in Figure 5 and Figure 6. Removing this anchorage entirely yields no structural advantage. The strip seal anchorages used by the Iowa DOT are not required to be bolted to the top flange of the girder. Thus, if nothing will be bolted to the top flange there is little reason to remove the anchorage from the top flange. In the contractor's experience it took significantly less time to simply cut out any sections of the old anchorage that are in the way of the new joint and leave the rest embedded in the deck where it is already out of the way. More so, if a joint replacement was combined with a deck overlay, only a few inches of deck may need removed. If the removal is sufficiently shallow, it may be possible to avoid the issue of removing concrete from around the longitudinal bars.

Figure 5 – Sliding Plate Joint Anchorage During Removal

Figure 6 - Sliding Plate Joint Cross Section

The removal of deck rebar may also greatly speed the pace of demolition. The contractor estimated that if there is no requirement to maintain existing deck rebar, demolition time could be halved. Rebar is particularly tricky to maintain on skewed bridges. On a non-skewed bridge, the right image in Figure 7, transverse bars are easily removed and replaced during construction as the surrounding concrete will be entirely removed. On a skewed bridge, the left image in Figure 7, transverse bars must be maintained in good condition as well as the longitudinal bars. Not only does this require twice as much reinforcing to be maintained without damage, but it is also twice as much material to work around to remove concrete and the existing joint anchorages.

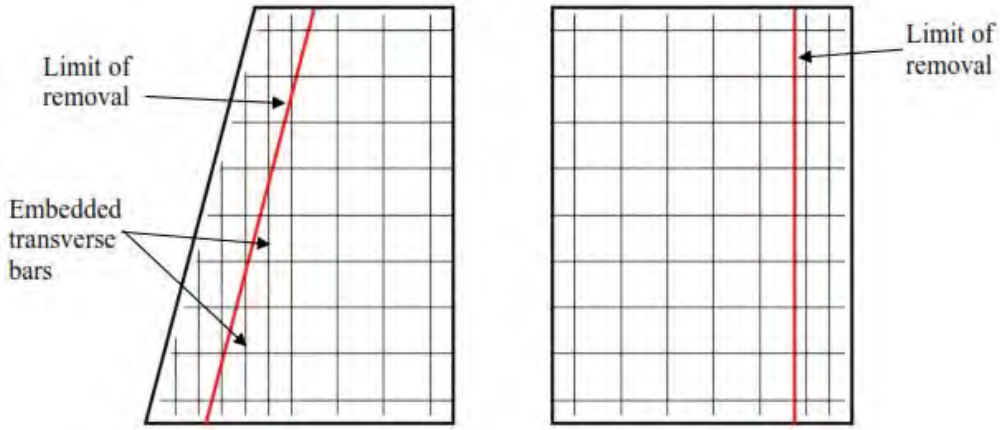


Figure 7 - Skewed Bridge Deck Reinforcing (left). Non-Skewed Deck Reinforcing (right).

For a typical joint replacement, dowels could be drilled horizontally in the bridge deck and the bars could be replaced in a fraction of the time saved in the demolition phase. However, if a partial depth removal is

used and the rebar does not need to be replaced, there would be no additional time spent replacing these bars anyway. According to the contractor drilling holes for rebar is rapid and efficient for horizontal bars up to 1 ft in depth and for vertical bars up to 1.5 ft in depth.

The debate between actually using a precast section and a cast-in-place section also occurred in Group B. In particular Group B's discussion involved how the joint extrusion was actually set in place and aligned. Since metal joint extrusions typically have a clearance between the top and bottom of the concrete header, they do not actually bear on the bridge deck at any point and cannot be attached directly to the deck for alignment. Typically they are aligned and held in place by hanging the joint from a series of angle iron supports that span over the replacement joint. These hangers must then be removed after the concrete header has been poured but before the concrete has set. If a precast section were used set screws could be used to adjust the elevation of the joint. Set screws would be a simpler method of aligning the joint, but would still require some grout to both cover the set screws, and to seal the gap between the new and existing sections. Thus, despite using a precast section there will still be some cementitious material required to at the least to seal the construction joint. However, a precast section would allow higher strength concretes to be used as a joint header potentially increasing joint durability.

Cast in place concretes can also obtain high strengths in a short period of time, but these materials may not be readily available in rural areas.

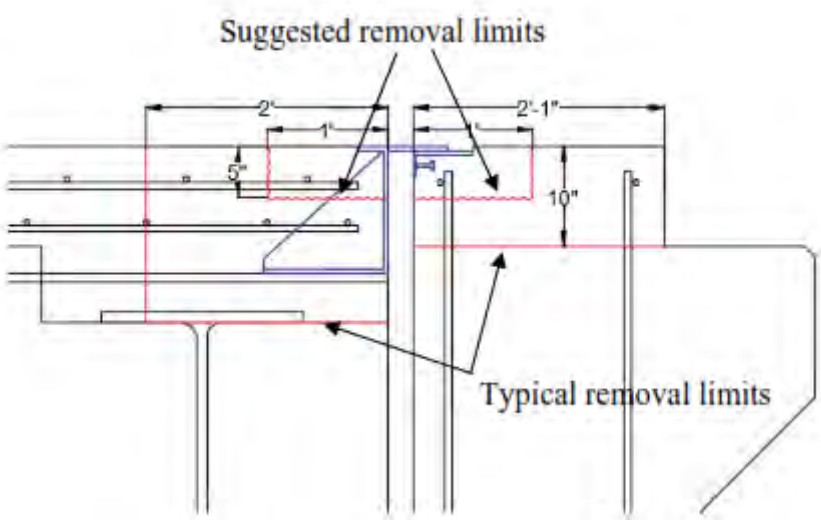


Figure 8 - Suggested Demolition Limits for Partial Depth Joint Replacement

The general detail developed by Group B is shown in Figure 8. The simple explanation is that this detail involves doing less work. The first step is to simply remove less concrete. Remove only the concrete and

extrusion necessary to anchor a new joint. The rest of the existing anchorage can just be left embedded in the bridge deck. The suggested removal limits in Figure result in almost a 60% reduction in the amount of concrete removed from the typical removal limits. Since demolition is usually the longest task in a joint replacement this alone should greatly expedite the rate of joint replacements. Allowing the removal of the reinforcing that protrudes into the demolition areas would also speed up construction. There are a few issues that need investigated before these bars are removed. First, are 1 ft or less of the end sections of these longitudinal bars (transverse bars are easily replaced) actually necessary for the structure of the bridge deck. Will the end of the deck perform as intended without these sections? Second, can the new concrete header bond well enough to the exiting concrete that these bars will not be needed to connect the new and existing sections. In this regard, elastomeric concretes have performed well, but there are other concerns with their usage. Cement based concretes, UHPC, HPFRC, high early strength etc., may be better for anchoring extrusions but their ability to bond to an existing section without the use of anchors is uncertain. If anchors are required it would be possible to remove the existing rebar, and quickly add anchors to provide composite action for the concrete. Several types of anchors, including drilled and chemically bonded (polymer grouted), are regularly used by the Iowa DOT. Caltrans (2) suggests avoiding mechanical expansion anchors to resist tensile forces where concrete is likely to crack or to resist dynamic loads, both concerns near the expansion joint. When using chemically bonded anchors high quality control must be ensured. These same anchors may also require holes greater than 10 inches to develop their full tensile strength. A hole this deep should not be problematic as the joint will typically be installed at the top of a diaphragm on one side and at the top of the abutment on the other.

The last necessary item that will need to be addressed is the joint anchorage. The standard joint anchorage detail utilized by the Iowa DOT shown in Figure 9 is quite large. The total height of the joint and anchorage is almost about 6.5 inches tall and nearly 13 inches long. This anchorage currently needs more concrete removal than the suggested limits. However, this anchorage is properly functioning with failures of the metal extrusion usually occurring at the weld between the extrusion and the anchorage as discussed with DOT maintenance forces during the investigation of deterioration patterns and maintenance efforts. Thus, a new design would require a design that is at least as robust as the existing anchorage, but with a smaller profile. The authors suggest redesigning the anchorage to allow it to be attached to drilled and chemically bonded anchors installed at the end of the bridge deck. These anchors, as discussed above, could also serve the dual purpose of providing a bond between the new and existing concrete allowing for the removal of the existing reinforcing.

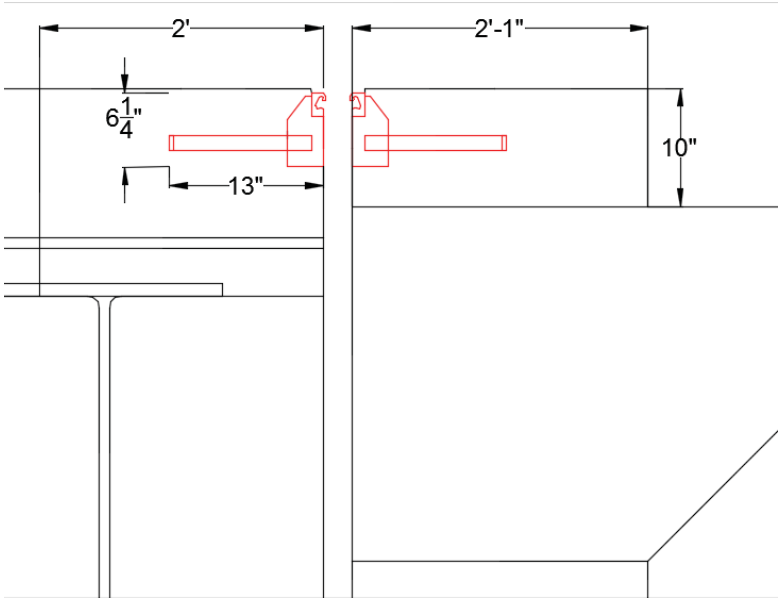


Figure 9. Iowa DOT strip seal anchorage

Workshop Conclusion

The two proposed concepts differ in the mindset behind them. Group A, deck sliding over backwall, has suggested an approach that may not accelerate the project, but will hopefully result in a better end product. Group B, replace less, would hopefully accelerate the project but the same styles of joints will be going in the same places. Both ideas may be appropriate in the right places. The suggestions proposed

by Group B will take less time, cost less than Group A, but should also be expected to last no longer than a standard strip seal expansion joint would last. Group A on the other hand is intended to improve the bridge beyond its initial design, but by taking longer with an inevitably more expensive design. However, the deck sliding over the backwall design was intended to last, with proper maintenance, until the bridge is eventually replaced. Both concepts have their place in meeting the challenges of managing the bridge infrastructure at different points of their life-cycle.

CONCLUSIONS

The research yielded the following conclusions:

- Demolition and concrete cure times are the longest segments of expansion joints replacement projects. The largest percentage of time would be saved by reducing these steps. However, all steps tend to tie together with embedded reinforcing largely controlling the length of the demolition stage. Requirements to maintain rebar in good condition necessitates the usage of smaller handheld demolition equipment as opposed to larger tractor mounted breakers that would

damage the embedded reinforcing. It was thought by contractors on the TAC committee that removal of existing rebar and installation of dowel bars would be faster than maintaining the rebar. However, this introduces concerns with spalling of the bridge deck if cover concrete is not of sufficient depth. Therefore, removal times can be affected by affecting the arrangement of reinforcement.

• On one observation hydrodemolition was used in lieu of handheld pneumatic breakers for demolition of existing concrete. With experience it appeared that hydrodemolition could be a more rapid process than pneumatic breakers. However, data to this regard could not be gathered at the time. Several drawbacks existed with hydrodemolition including the need for an enormous quantity of water, a considerable amount of runoff including small particulate, as well as the need for some pneumatic removal in inaccessible regions. It also required the use of expensive equipment not commonly used by contractors in the state of Iowa for this purpose.

• Repair of expansion joints is being done as needed, but preventative maintenance is largely ignored. Cleaning of sealed expansion joints to remove collected debris is only performed if other repairs are being completed on the same bridge. Additionally maintenance technicians have observed that neoprene glands perform well up to 15 years and 10 years for strip seal and compression seal joints respectively. Performance of the neoprene seals after this point can be unpredictable with seal replacements waiting until failure has happened. During the time from which failure is discovered until it is finally replaced the joint is left open for the possibility of damage to occur. An alternative would be to replace seals at 15 and 10 years respectively before they fail.

• Emergency repairs of legacy type joints, often sliding plate joints, by Iowa DOT personnel typically consist of doing whatever is necessary to allow the movement of the bridge deck and the passage of traffic. Restraints on time, manpower, and materials prevent repairs from improving the joint to a better working condition. Joints are left leaking with a rough riding surface. Several types of joints exist that require very little installation time including adhesive bonded joints and expandable foam compression joints (e.g. EMseal joint). These joints could be used to provide temporary waterproofing until a full joint replacement can be completed. However, doing so requires these joint materials to be stockpiled so they are readily available when unexpected emergency repairs occur.

• Waterproofing is an important function of expansion joints in Iowa to prevent substructure damage from corrosives. With the joint performing such a critical function, redundancy in waterproofing could prevent damage to the substructure in cases where the joint has undergone damage but is not yet slated for replacement. As damage happens, such as damage to neoprene glands where the watertight seal has been broken, the expansion joint will prevent the passage of most debris while the trough will still prevent water and dissolved corrosives from damaging substructure components.

• Full removal of old sliding plate joint anchorages are unnecessary during joint replacements. The old anchorages were typically bolted to the top flange of the steel girder and require a considerable amount of concrete removal, time and effort to remove. Alternatively the concrete can be removed to the depth required for the new strip seal anchorage and the exposed sections of the sliding plate joint anchorage can be removed with a cutting torch. The rest can be left embedded in the existing concrete.However, it must be ensured that structural capacity for the strip seal anchorage is still met.

SUGGESTIONS FOR FUTURE RESEARCH

• Develop a suitable high-early-strength concrete mix to be used in repair applications. Alternatively analyze existing commercial products developed for this purpose to achieve a successful mix. Both pre-bagged mixes that could be stockpiled and stored for emergency repairs on short notice as well as large batched mixes order from concrete batch plants should be considered. Prior research has found that concrete strength requirements can easily be met in as little as 4 hours, but that these mixes often suffer from increased amounts of shrinkage which can cause premature deterioration in repair projects.

Alternatively other types of concretes exist including polymer concretes and magnesium phosphate based concretes, each capable of achieving high-early-strengths.

• Redesign strip seal anchorages for a smaller profile. Current anchorages used in Iowa are nearly 6 inches in depth usually requiring at minimum the removal of the full-depth of the bridge deck to install a new joint. A smaller profile could reduce the amount of concrete required to be removed, particularly if coupled with a bridge overlay, which could reduce the amount of reinforcing that needs exposed. However, it was noted that current anchorages are effective with joint damage due to anchorage pullout rarely occurring.

• Design, instrument, and observe a “deck sliding over backwall design”. Discussions during the two workshops completed as part of this research indicated that it would be a superior design to move the joint away from the bridge deck. It was also thought that such a repair could also be completed in a single weekend which would not reduce the time required for joint replacements, but would create a more effective joint in the same amount of time. Experiences with a similar type of repair in Michigan was indicated to be positive.

ACKNOWLEDGMENT

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CFT BRIDGE PIER CONNECTIONS FOR ACCELERATED CONSTRUCTION IN SEISMIC REGIONS

Max T. Stephens[1], Dawn E. Lehman[2], Charles W. Roeder[2]

Concrete filled tubes (CFTs) are composite structural elements which provide large strength and stiffness while permitting accelerated bridge construction (ABC). The steel tube serves as formwork and reinforcement to the concrete fill, negating the need for reinforcing cages, shoring, and temporary formwork. In relation to ABC, the placement of the concrete fill may be further enhanced using self-consolidating concrete (SCC), so that concrete vibration is not required.

Although CFTs offer many advantages in rapid construction and structural performance, connections between CFTs are often different and more complex than those used in steel or reinforced concrete construction due to the composite nature of CFTs. The focus of this research is the development and experimental investigation of robust CFT column-to-cap beam connections capable of sustaining cyclic lateral loads while minimizing damage and degradation. The study focus is on precast bent caps, since this benefits ABC, and practical design expressions are developed for these connections based upon the experimental research.

Proposed Connections

The proposed CFT column-to-cap beam connections are illustrated in Fig. 1. There are three connection types: (1) embedded ring connections (Fig. 1a), (2) welded dowel connections (Fig. 1b), and (3) reinforced concrete connections (Fig. 3c). This provides a suite of connections for designers, each option offering advantages as the project may require.

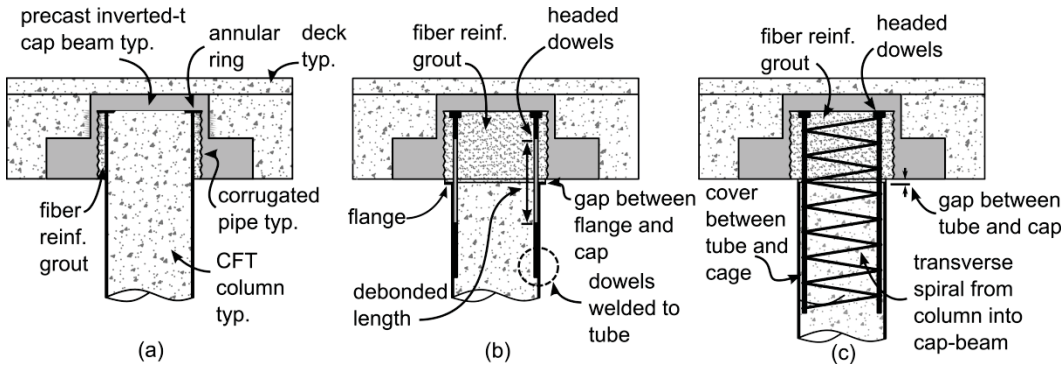


Fig. 1 Proposed CFT Column-to-Precast Cap Beam Connections. (a) Embedded Ring Connection (ER), (b) Welded Dowel Connection (WD), and (c) Reinforced Concrete Connection (RC)

Fig. 1a shows a full strength embedded ring connection (herein referred to as ER) which uses a grouted connection detail, with a void cast into a precast cap beam. A circular ring is welded to the steel tube to provide anchorage and transfer stress to the concrete and reinforcing in the cap beam. The precast cap beam is placed onto the column after the column is set, and the recess between the tube and corrugated pipe is filled with high strength fiber reinforced grout.

Fig. 1b shows a welded dowel connection (herein referred to as WD). The WD connection utilizes headed dowels to resist the flexural demand. The shear transfer to the tube is accomplished by welding the dowels to the steel tube using a flare bevel groove weld. Welding the dowel directly to the tube, as opposed to embedding the dowel directly into the connection maximizes the moment capacity of this connection. A gap is included between the steel tube and cap beam. A flange is welded to the exterior of the steel tube to increase compressive bearing area on the soffit fill as illustrated in Fig. 1b. The dowels can be de-bonded in the column-to-cap beam interface region to increase the deformation capacity of the connection.

Fig. 1c shows a reinforced concrete connection (referred to as RC connection) in which a short independent cage for both transverse and longitudinal reinforcing extends from the CFT column into the cap beam, and cover is provided between the reinforcing cage and steel tube within the column. A gap is left between the steel tube and cap beam to help focus the plastic hinging location between the CFT component and the cap beam.

EXPERIMENTAL PROGRAM AND RESULTS

Eight large scale specimens were designed to experimentally evaluate the performance of the proposed connections under constant axial and reversed cyclic lateral loading. The scale of the specimens ranged from 50% - 60% based on a prototype bridge located in Santa Rosa, California. For the facilitation of

ABC, all of the connection specimens used a grouted detail, with a void cast into a precast beam as shown in Fig. 1.

The moment-drift responses of three selected specimens are shown in Fig. 2, while the final states of these specimens are illustrated in Fig. 3. The moments have been normalized by the theoretical strength of the CFT column as calculated using the plastic stress distribution method (PSDM). All of the specimens exhibited large strength and ductility while limiting damage to the cap beam. Damage states and failure modes observed during testing are summarized here for the selected specimens.

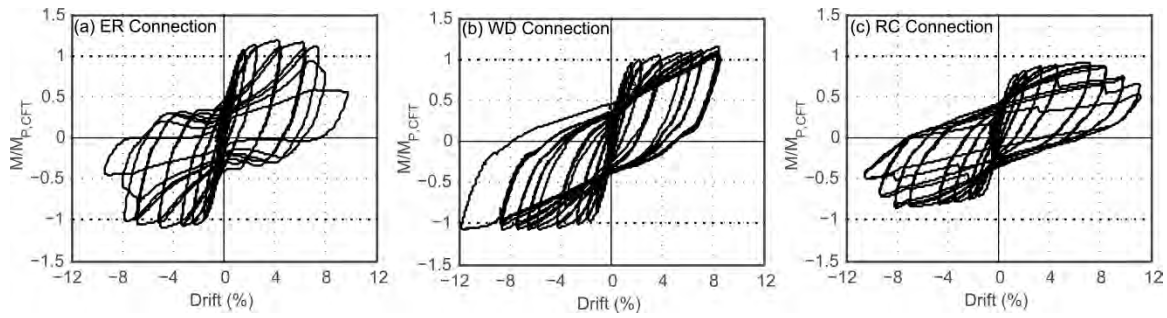


Fig. 2 Moment-Drift Response

Embedded Ring Connection

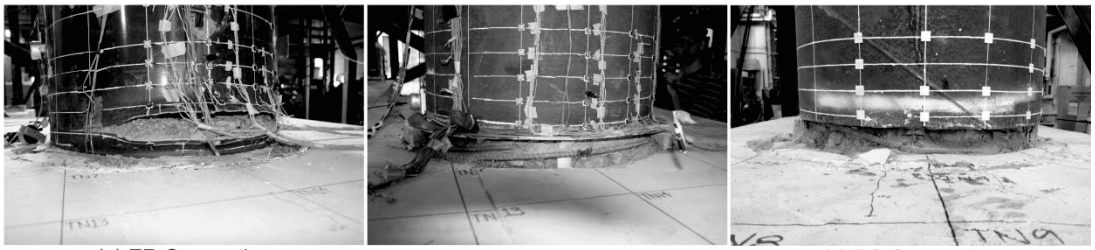
Tube buckling was observed at 3.5% drift, however no strength degradation was observed. Tube tearing initiated at 7.5% and the test was stopped at 9% drift as tube tearing propagated around the base of the column. The ultimate moment achieved using this connection was larger than the theoretical plastic capacity of the CFT column.

Welded Dowel Connection

The specimen was tested cyclically to 9% drift with no strength degradation or bar buckling observed. A monotonic push was conducted to 12% drift with no influence on strength. Very limited cap beam damage was observed. The ultimate moment achieved using this connection was larger than the theoretical plastic capacity of the CFT column.

Reinforced Concrete Connection

The specimen was cycled to 8.75% drift with no strength degradation. At 9% drift, reinforcing bars at the extreme fibers fractured. The remainder of the reinforcing bars fractured in subsequent cycles, and the test was stopped at 12% drift. Very limited cap beam damage was observed. The ultimate moment achieved using this connection was smaller than the theoretical plastic strength of the CFT.



(a) ER Connection (b) WD Connection (c) RC Connection

Fig. 3 Final States

DESIGN EXPRESSIONS

The experimental results and observations were used to develop practical engineering expressions for the proposed CFT column-to-cap beam connections. The connection should be designed as one of the following options:

1. An ER connection in which the CFT column is embedded into the cap beam as illustrated in Fig. 1a.
2. A WD connection in which a ring of partially debonded vertical headed reinforcing bars are welded inside the CFT column and extend into the cap beam as illustrated in Fig. 1b.
3. A RC connection in which a ring of headed reinforcing bars is developed into the steel tube and extend into the cap beam as illustrated in Fig. 1c.

Each of these options can be employed using cast-in-place (CIP) or precast super-structure cap beam. The specific design expressions for each connection type have not been included here for brevity, but are readily available in reference material.

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FIELD-CAST CONNECTIONS FOR PREFABRICATED DECK PANELS: PERFORMANCE OF PRE-BAGGED CONNECTION GROUTS

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INTRODUCTION

Prefabricated deck panel systems are one of many prefabricated bridge element (PBE) types that have gained popularity for both new construction and rehabilitation/re-decking projects. Many of these systems relay on interlaced connector elements and field-cast pre-bagged grouts to create structural continuity between adjacent deck panels and the supporting girder system. The ideal grout for PBE connections grout would be self-consolidating, have high early strength, and have good durability and dimensional stability. The mechanical and durability properties of these grouts directly impact the short- and long-term performance of prefabricated bridge deck systems. Currently, there are a number of different products available in both U.S. and international markets that possess many of the desired aforementioned properties. There have been studies that compare the material-level behavior of different candidate connection grout materials (1, 2), but limited data is available comparing the system-level behavior of connections with different grout materials. An on-going research project at the FHWA Turner-Fairbank Highway Research Center (TFHRC) is focused on comparing the performance of different pre-bagged grouts in prefabricated deck connections. To date, a total of 74 large-scale precast deck panel connection tests have been conducted. The primary goal of this experimental program is to determine how different connection grouts behave with different combinations of deck panel connection details. A number of parameters frequently considered during the design of these connections, including grout type, lap splice length, reinforcement detailing and type, and precast surface preparation were investigated. This paper will highlight the key points of this project.

EXPERIMENTAL PROGRAM

Figure 1 shows an illustration summarizing some of the test variables and provides some photos from specimen construction. Specimens were constructed by joining two precast deck panel halves using non-contact lap splices and a given connection grout material. To date, five different connection grout

materials have been evaluated including non-shrink cementitious grout with and without internal curing, magnesium phosphate grout, epoxy grout, and ultra-high performance concrete (UHPC) grout; four other UHPC materials are slated for testing in the near future. All panels were reinforced longitudinally with Grade 60 No. 5 bars. The reinforcement details and non-contact lap splice lengths varied for each grout type. Connections employing UHPC used straight 5.5-in lap splices similar to that shown in Figure 1. The majority of specimens employing non-UHPC grouts used straight 12-in lap splices; some panels did however employ lap splices of different length, headed reinforcement, or U-shaped reinforcing bars. A series of monolithic concrete panels were also cast to provide baseline measurements. Although not discussed herein, a series of small-scale beam tests were conducted to evaluate the bond strength between connection grouts and precast concretes with different surface preparations. Deck panel specimens were loaded in four-point bending, and were subjected to three different loading protocols. The loading protocols were designed to evaluate the resistance to cracking in the connection region under low-level cyclic loading, resistance to damage and degradation during moderate-to-high-level fatigue loading, and to determine ultimate strength of the connection under monotonic loading. These protocols are referred to "cyclic crack loading", "fatigue loading", and "ultimate loading", respectively. The protocols were applied in succession. If a specimen survived the fatigue loading protocol without significant loss of load carrying capacity it was subsequently subjected to monotonic loading until failure. Further details related to the loading protocols can be found in (3).

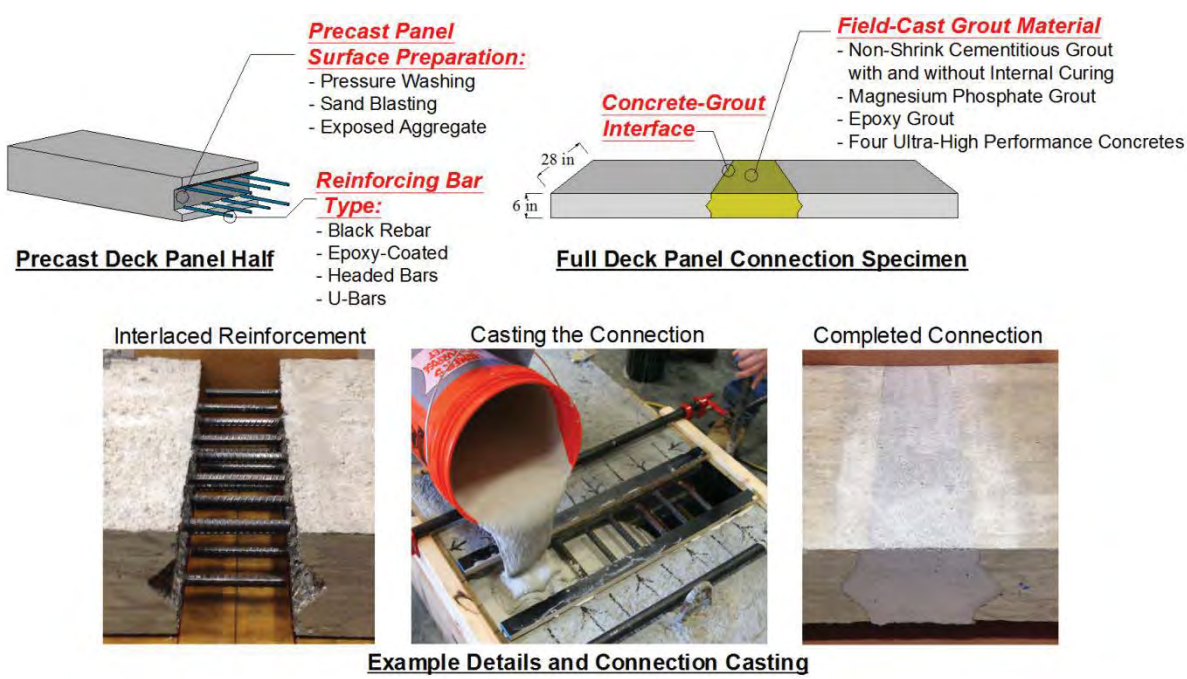


***Figure 1**. Test Variables, Example Details, and Casting Photos*

KEY OBSERVATIONS

Key observations are presented according to connection the grout material. Figure 2, which is presented at the end of the section, depicts the cracking damage observed in specimens after the cyclic cracking loading protocol. The conference presentation will expand on the key observations presented herein.

Connections Employing Non-Shrink Cementitious Grouts

Prior to testing, specimens were inspected for shrinkage cracking. Eighty-four percent (27 of 32) of specimens employing non-shrink cementitious grout exhibited visible shrinkage cracking in the connection grout. Shrinkage cracking (shown with blue lines) in a specimen with non-shrink grout can be observed in Figure 2. During cyclic cracking loading, shrinkage cracks tended to propagate in both transverse (flexural) and longitudinal (splitting) directions. Visual observations and data analysis indicated the bond strength between precast concrete and non-shrink grouts was weak regardless of surface preparation. Forty-four percent (14 of 32) specimens failed during fatigue loading as a result of splitting cracking, which led to bond failure between the reinforcing bars and grout. Specimens that did not fail during fatigue loading exhibited low to no displacement ductility during ultimate loading, and low strengths compared to the monolithic baseline panel. Panels employing headed bars and U-bar details, or panels with added confinement (from transverse bars) within the connection region exhibited improved ultimate loading behavior.

Connections Employing Magnesium Phosphate Grout

Specimens employing magnesium phosphate grout did not exhibit visible shrinkage cracking in the connection region. There was little to no apparent damage in the connection region after cyclic cracking other than cracking at the interface between precast concrete and the connection grout material. Similar to what was observed with specimens with non-shrink grout, the bond strength between precast concrete and the magnesium phosphate grout was weak regardless of surface preparation. All specimens failed during the fatigue loading protocol as a result of splitting cracks and bond failure between the reinforcing bars and grout.

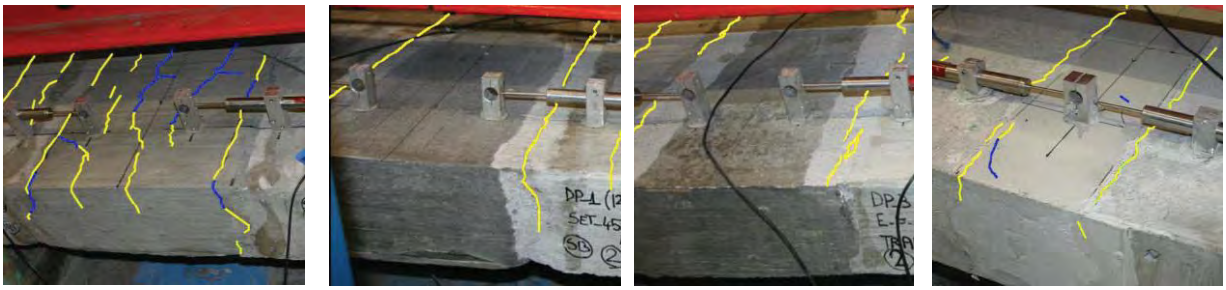
Connections Employing Epoxy Grouts

Epoxy grouted specimens did not exhibit shrinkage cracking, and there was little to no apparent damage in the connection region after cyclic cracking other than cracking at the interface between precast concrete and the connection grout. In general, the epoxy grout material exhibited very good bond between with precast concrete regardless of surface preparation. In some case, the interface was intact after cyclic cracking, and did not crack until fatigue loading. All specimens employing epoxy grouted connections survived fatigue loading protocol with minimal damage to the connection. Lastly, all specimens exhibited

good displacement ductility and ultimate loading capacities. In all cases, the ultimate performance of specimens with epoxy grouted connections was comparable to that of the monolithic baseline panel. Furthermore, failure was governed by concrete crushing.

Connections Employing UHPC Grouts

Thirty-three percent (6 or 18) specimens with UHPC grout exhibited shrinkage cracking in the connection. However, shrinkage cracks were fine and were very localized, which can be seen in Figure 2. There was little to no apparent damage in the connection region after cyclic cracking other than cracking at the interface between precast concrete and the connection grout material. The UHPC grout exhibited good bond strength to precast concrete when an exposed aggregate surface finish was used. All specimens employing UHPC grouted connections survived fatigue loading protocol with minimal damage to the connection. All specimens exhibited displacement ductility and ultimate loading capacities comparable to that of the monolithic baseline panel. Furthermore, failure was governed by concrete crushing.



Non-Shrink Cementitious Grout without Internal Curing | Magnesium Phosphate Grout | Epoxy Grout | UHPC Grout

Figure 2. *Observed Cracks in the Connection Region after Cyclic Cracking Loading; Blue Lines Denote Shrinkage Cracking, and Yellow Lines Denoted Cracking Caused by Cyclic Loading*

CONCLUSIONS

Based on the findings of this line of research, it can be concluded that the selection of grout materials is a critical design consideration for deck-level PBE connections. Although epoxy and UHPC grout systems have higher initial cost, they could provide better value when constructability, long-term performance, and required maintenance are considered. Conventional non-shrinkage and magnesium grouts, on the other hand, may lead to durability issues as a result of poor bond to precast concrete regardless of surface preparation and poor performance under repeated loading. Lastly, it is recommended that exposed aggregate surface preparations be used to promote bonding between precast concrete components and grout connection materials.

ACKNOWLEDGMENTS

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G-11: ABC RESEARCH

SLIDE-IN BRIDGE CONSTRUCTION COST ESTIMATION TOOL

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FOREWARD AND ACKNOWLEDGEMENTS

The Slide-In Bridge Construction (SIBC) Cost Estimation Tool was developed for the Federal Highway Administration's (FHWA) Every Day Counts Initiative to promote the use of this innovative Accelerated Bridge Construction (ABC) technique to shorten project delivery, reduce user impacts, and enhance roadway safety. The tool provides a general guideline for state Departments of Transportation (DOTs) to estimate the cost of SIBC for common bridge replacements.

Leidos, Inc. and Michael Baker International developed the SIBC Cost Estimation Tool.

Technical Working Group contributors include Iowa DOT, Minnesota DOT, Oregon DOT, Utah DOT, and Wisconsin DOT. Additional contributors of project data include Colorado DOT, Indiana DOT, Michigan DOT, Missouri DOT, Nevada DOT, New York DOT, Oklahoma DOT, Oregon City, and Washington DOT.

PROJECT OVERVIEW

Slide-In Bridge Construction (SIBC) is a relatively new, innovative, and underutilized technology. The tool implements historical cost data to facilitate the preparation of construction cost estimates for projects using SIBC technology.

Slide-In Bridge Construction Overview

SIBC offers a cost-effective technique to rapidly replace an existing bridge while reducing impacts to mobility and increasing safety. SIBC is an ABC technology that reduces the on-site construction time associated with building bridges.

SIBC allows for construction of a new bridge while maintaining traffic on the existing bridge. The new superstructure is built on temporary supports adjacent to the existing bridge. Once construction is complete, the road is closed, the existing bridge structure is demolished or slid to a staging area for demolition, and the new bridge is slid into its final, permanent location. Once in place, the roadway

approach tie-ins to the bridge are constructed. The replacement time ranges from overnight to several weeks. A variation of this method is to slide the existing bridge to a temporary alignment, place traffic on the temporary alignment, and construct the new bridge in place

SIBC provides an effective alternative to phased construction, crossovers, lane reductions, or use of temporary bridges. Although the lateral slide requires a short-term full closure of traffic, owners and the public typically prefer the limited impacts of a single short-term closure when compared to the extended traffic impacts associated with phased construction.

BACKGROUND

Data Collection

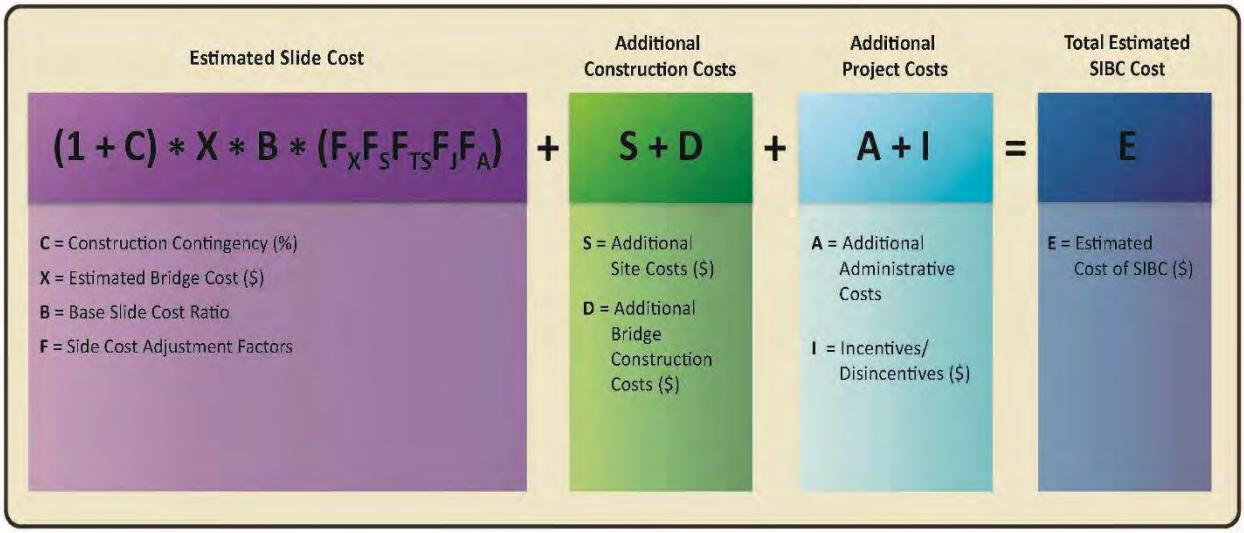
The SIBC Cost Estimation Tool incorporates project data from 29 completed SIBC projects nationwide. These projects represent all completed SIBC projects for state DOTs to date.

The quantity of projects and available data provided somewhat limited data. For example, seven of these projects were delivered using Design-Build contracting and detailed cost breakdowns were not available. However, the detailed cost data from the other 22 projects provided significant correlations, which were used to develop this tool.

The project also utilized a technical working group that included the project team and representatives from four of the state agencies that provided project data. This group reviewed the tool and provided significant input in its development.

Methodology

The tool estimates the additional cost of using an SIBC alternative by factoring the estimated construction cost of the bridge to determine the slide cost and then adds associated construction and administrative costs. This method is user-friendly as it establishes a clear relationship between the inputs and calculations performed in the spreadsheet. The cost adjustment factors are calibrated primarily on historical data. Some adjustments based on experience were necessary when there was insufficient historical data.



COST ESTIMATION

Total Estimated SIBC Cost

The term E, Total Estimated SIBC Cost, represents the entire additional (or delta) cost of an SIBC project as compared to traditional construction. This term captures both construction and administrative costs to provide a holistic project cost difference. To obtain the total cost of the SIBC alternative, simply add the E value to the project cost of the traditional construction alternative.

The SIBC method offers many distinct benefits to the agency, namely reduction in traffic impacts and increased safety. Under certain circumstances, SIBC can also reduce overall project costs. In these situations, the E term will be negative, which indicates an overall estimated project cost savings.

Estimated Slide Cost

This portion of the methodology equation estimates the cost for the contractor to build temporary supports and slide the bridge into place. It includes the following:

- Temporary supports, including foundations, platform ("table top"), bracing, etc.
- Equipment rental such as jacks, guides, tracks, monitoring equipment, etc.
- Vertical jacking
- Production of shop drawings, move schedules, and monitoring plan by the contractor
- Labor

Many agencies choose to use a lump sum line item in the construction bid for this work – this term estimates that cost. When the tool is used to estimate only the slide construction costs, such as for construction bid evaluations, the S, D, A, and I terms should be zero.

The user determines the contingency. The user is encouraged to input values in the tool that are common practice for the associated agency.

The X term is the estimated bridge construction cost for the structure that will be slid and should not include any estimated slide costs. During the planning phase, this term may be a simple estimation such as deck area multiplied by a unit cost. During the design phase, this value should be the engineer's bridge construction cost estimate (without any slide costs). This term should not include embankment, roadway tie-in, or adjacent retaining wall costs.

The B term is the average ratio of slide cost to bridge cost. This value, 20 percent, is empirically based and is not changed by the user. The value was determined using the winning contractor bids from previous construction projects.

Several site specific factors can increase or decrease the cost of a bridge slide. The cost adjustment factors used in this tool include:

- Experience Factor
- Site Complexity Factor
- Temporary Shoring Factor
- Vertical Jacking Factor
- Annual Average Daily Traffic (AADT)/Undercrossing Factor

Other factors were considered but were not included due to the lack of a definitive relationship.

Additional Construction Costs

The terms S and D account for additional construction costs that are not included in the slide cost. The S term accounts for additional site costs such as ROW or utility relocations. The D term accounts for additional bridge costs such as girder type or span length. These cost items are physical items that become the responsibility of the agency. These items may provide ancillary benefits to the agency.

Additional Project Costs

The terms A and I account for additional project costs that are required for SIBC. The A term accounts for additional administrative costs such as agency oversight, design engineering, and construction engineering, and inspection. The I term accounts for any additional incentives or disincentives for the contractor. These terms represent non-physical costs.

PRECAST BRIDGES UNDER VERY HIGH HORIZONTAL LOADS

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INTRODUCTION

It's common to associate precast bridge deck technology with small and medium span solutions under important vertical loads but reduced horizontal (longitudinal and transversal) forces. Consequently precast structures have been usually forgotten or even rejected in geographical regions with important seismic activity. It is wrongly supposed that precast construction processes introduce some uncertainties in the dynamical behavior of the structure. In high-speed railway bridges the design horizontal loads are, by far, the most important ones, and precast technology has shown its capabilities with no related problems.

A 173 meters long high-speed (designed for 350 km/h=217 mph) railway precast deck is presented. The span distribution is 28.5+36+48+34+26. The design process has been fully aware of the problems related with the transmission of horizontal loads, basically the acceleration and braking force, through the connections between different precast elements. In the same way the dynamic effects involved (resonance phenomena and reduction of vertical accelerations to improve passengers comfort) have been deeply analyzed.

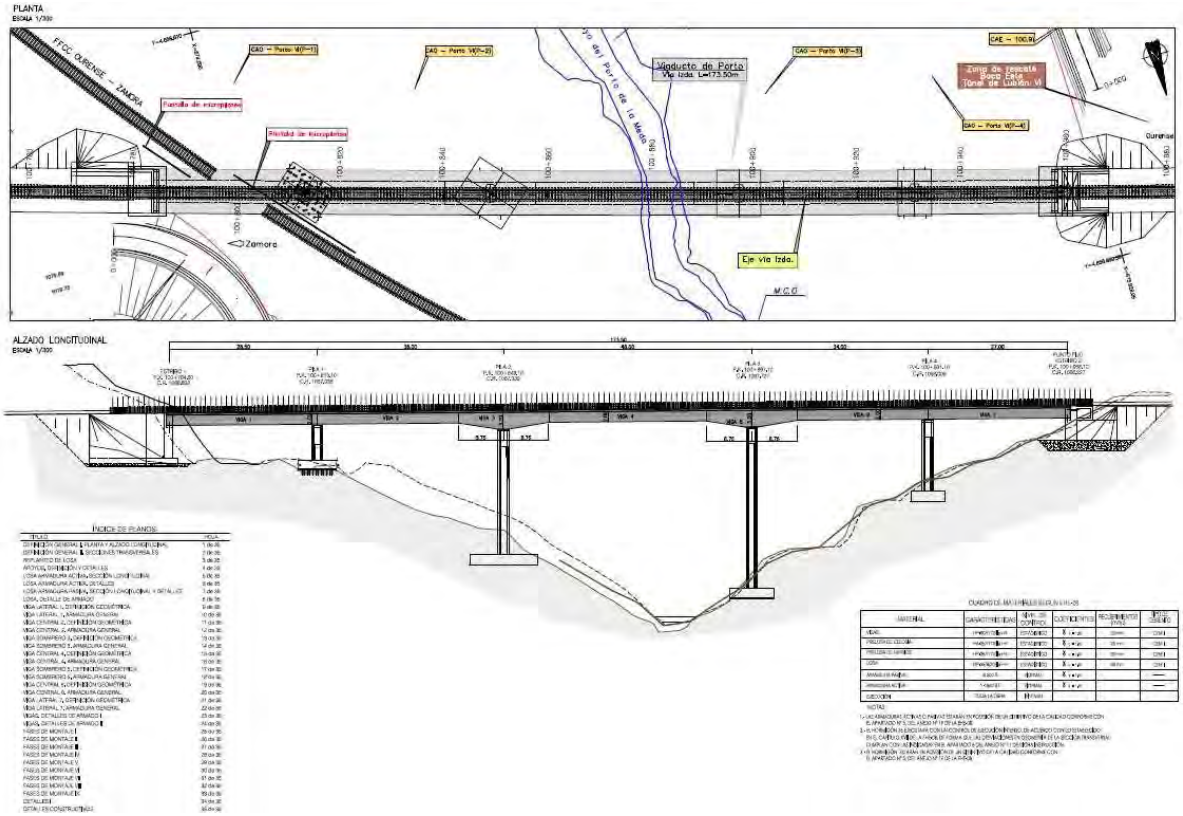

Figure 1: Plan and Elevation View of the Bridge

HISTORICAL BACKGROUND

In Spain precast concrete industry has been traditionally very important and well-developed; therefore the opportunity of participating in building the whole high-speed railway lines net was impossible to avoid.

The usual restrictions in cost and time budgets helped in choosing precast procedures as one of the best possible solutions for small and medium span bridges, not only for the deck but also for the substructure elements.

From the first high-speed railway line opened, in 1991, a lot of precast bridges have been built with optimal results, not only from a technical point of view but also in terms of cost control, quality and durability. It has to be noticed that one of the major concerns in high-speed railway engineering is durability and its influence in operating costs.

GEOMETRY

As said the span distribution is 28.5+36+48+34+26. The deck is built with a beam-and-slab system than can be thought as a simple combination of one beam spanning longitudinally and slab spanning

transversally. For longitudinal bending the slab acts as top flange of the beam (already working as a box) and for transversal behavior there's a combination between the flexural stiffness of the slab and the torsional stiffness of the complete box (beam and slab working together).

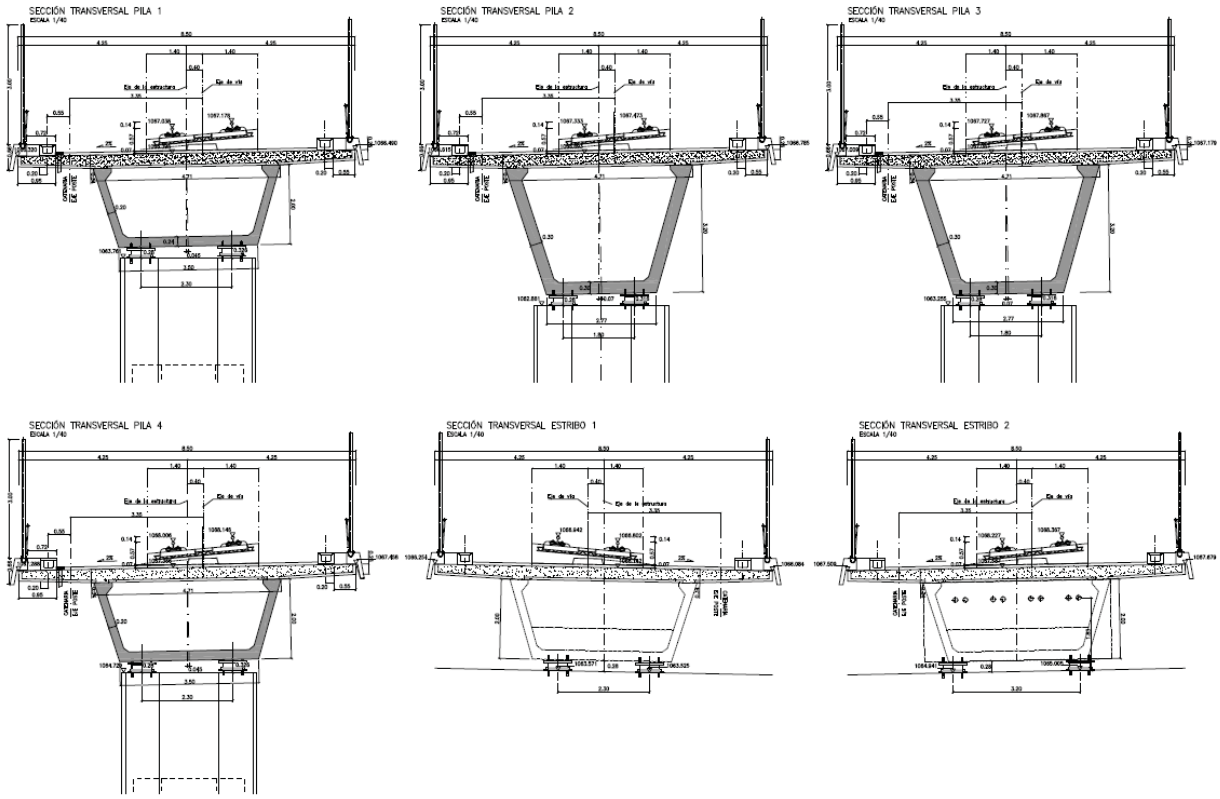

Figure 2: Deck Sections

The box beams depths vary linearly between 2.00 and 3.20 meters (6.50 and 10.50 feet) and their weight varies between 130 and 160 Tons. Over the beams, a post-tensioned in-situ concrete slab with varying thickness (26 cm at the slab extremes and 34 cm over the beam webs) is poured over reinforced pre-slabs which avoid any kind of scaffolding for the 8.50 meters (27.90 feet) wide deck. The resistance of the beams concrete reaches to 60 MPa (8700 psi) and the pre-slabs to 35 MPa (5100 psi). The poured concrete resistance is also 35 MPa.

The precast elements are post-tensioned (number 3, 4 and 5) or prestressed (number 1, 2, 6 and 7) concrete beams and the already mentioned reinforced pre-slabs. The beams are connected with short

(2.00 meters long) post-tensioned bars or cables (about 10 meters long), depending also of the relative position of the joint. These joints only achieve local sectional connections and, combined with the own

prestressing force of each one of the precast beams, are capable of ensuring the right behavior of the whole deck.

After pouring the concrete over the pre-slabs, the slab is post-tensioned through some ducts located in the concrete and anchored to some specific pre-slabs.

The design speed corresponds to high-speed standards, 350 km/h, which means that dynamic calculations control the structural response to a maximum speed of 420 km/h (20% greater than design speed)

CONSTRUCTION PROCESS

The building process is extremely challenging because the impossibility of accessing to the bottom of the valley for using cranes. And alternative procedure of beam launching gantry and consequent concrete deck pouring span-by-span has been finally chosen with optimal results.

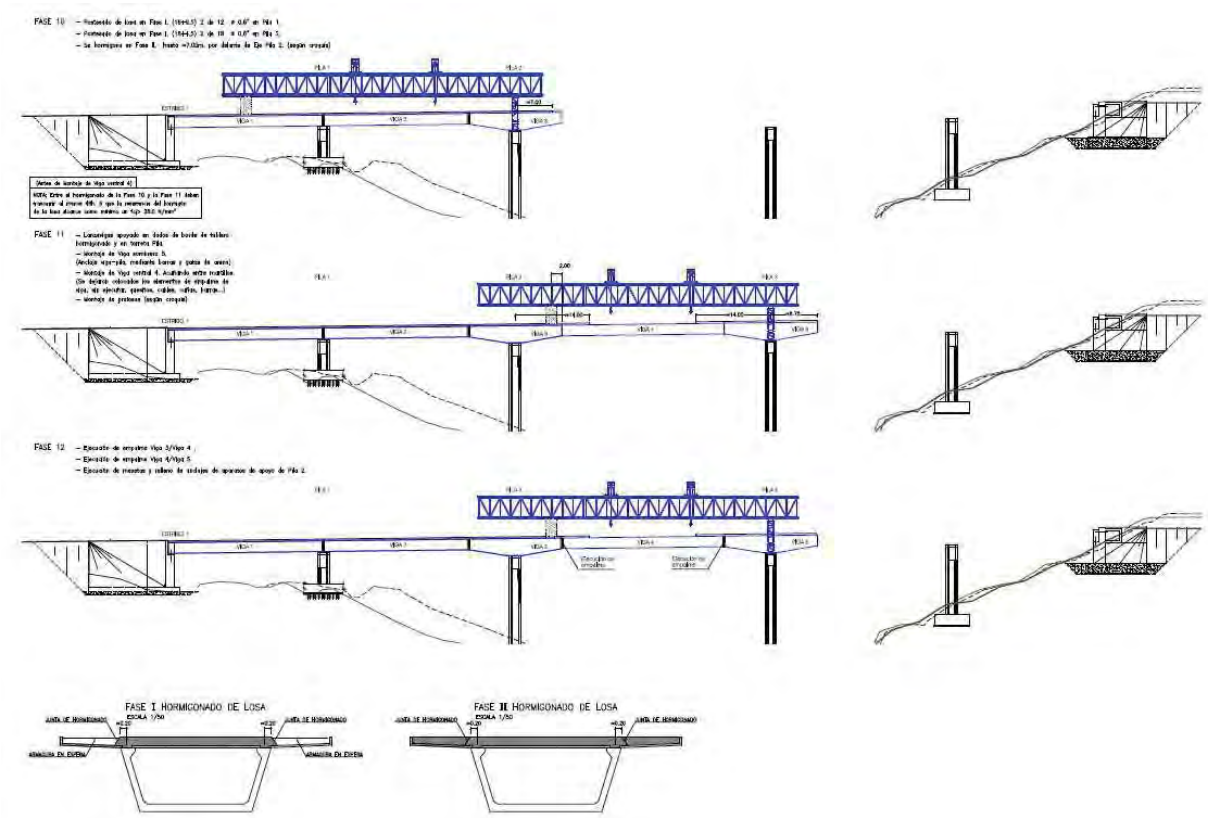

Figure 2: Phase of Construction Process

EFFECT OF INTERFACE MOISTURE CONTENT ON THE BOND PERFORMANCE BETWEEN A CONCRETE SUBSTRATE AND A NON-SHRINK CEMENT-BASED GROUT

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ABSTRACT

An increasing amount of bridge construction in the U.S. is completed through the use of prefabricated bridge elements (PBE), commonly relying on field-cast grout-type materials to complete the connections between precast concrete elements. The interface bond between the grout and the substrate concrete can be a key factor in the long-term durability of the structural system. This paper evaluates bond performance of a non-shrink cementitious grout, and examines how the supply of extra moisture at the grout-concrete interface affects the bond strength. The results show increased bond strength when supplemental moisture is provided to the substrate interface.

INTRODUCTION

The connection of prefabricated concrete elements using field-cast “non-shrink” cementitious grouts is a common practice in accelerated bridge construction (ABC) projects (1). The grout material should have sufficient strength and should offer good bond to the concrete element in order to insure adequate stress transfer not only during loading of the structure, but also during expansion and/or contraction of the newly placed grout material. However, recent studies have reported dimensional stability concerns (primarily shrinkage) in these grout materials (2), which could lead not only to durability problems of the grout material but also to the reduction of the bond between the grout and the prefabricated concrete element.

Many are the variables that affect the bond performance of a cementitious material (e.g., grout) when it is placed in contact with another cementitious material (e.g., concrete substrate) (3). One of them is the provision of extra moisture at the concrete surface before the pour of the new material. This is done with the goal of achieving a saturated-surface dry (SSD) condition on the substrate surface. It is hypothesized that the presence of this extra moisture to achieve an SSD condition will reduce the moisture transfer that

might occur from the fresh material into the concrete substrate, thus allowing the fresh material to use of all its available mixing water for a better hydration, as well as reducing shrinkage derived from the water migration. Shrinkage in the freshly poured material will not only increase the “gap” between the two materials, but it will also induce shrinkage stresses at the interface, typically causing microcracking. Since this practice of achieving an SSD condition by adding extra moisture has become common in the construction industry (especially in repair applications), this paper focuses on evaluating the effect that the supply of that extra moisture at the grout-concrete interface has on the bond performance.

EXPERIMENTAL

The bond assessment was performed using the ASTM C1583 test method (“pull-off” test method) on a grout-concrete slab (Figure 1). The slab dimensions were 36 in x 36 in x 4 in (914 mm x 914 mm x 102 mm) with a 2-inch (50-mm) thick overlay of a non-shrink cementitious grout. Prior to the grout pour, the top surface of the concrete slab was pressure washed at 24 h after casting in order to create an exposed aggregate interface, achieved by using a commercially available in-form paint-like retarder agent. The grout was cured for 2 d or 14 d prior to execution of the bond tests. The results are then presented as “2d” or “14-d” bond strength which refers to the age of the grout when the bond test was performed.

In this study, the moisture is provided by either saturating the concrete surface during the 24 h that precede the casting of the grout, so that an SSD condition is achieved, or by means of internal curing (IC) through the use of pre-wetted light-weight aggregates (LWA) being included in the grout material. The

LWA used had a specific gravity value of 1.57, and a water absorption value of 16.6 %. The amount of

LWA added was 23 % of the solid content, by mass. Water from the LWA will be released at the appropriate time (typically after set), and will theoretically migrate to the regions where water is demanded (e.g., grout-concrete interface). The provision of IC using LWA, besides providing extra moisture at the interface, will also reduce shrinkage of the grout and improve grout curing conditions (4). Finally, images taken from an environmental scanning electron microscope (SEM) will be utilized in order to observe if the extra moisture provided has an influence on the microstructural features (e.g., main hydration products formed) of the grout-concrete interface.

RESULTS

Figure 1 shows the 2-d and 14-d pull-off results. All the specimens failed at the grout-concrete interface (except for the tensile results in which the specimens failed within the grout material). Results labelled as “control” correspond to the specimens where no moisture was added (i.e., drying conditions at a

temperature of 23 °C ± 1 ºC and a relative humidity of 50 % ± 5 %). “SSD” and “IC” correspond to the specimens where additional moisture was provided via 24-h water saturation or IC, respectively. The pull-off tensile strength of the grout material for all the specimens was added or reference purposes. As can be observed, the bond strength increases over time, since the 14-d strength is (in all cases) larger than at 2d. The bond strength of the SSD specimens is about 45% and 17% higher than that of the control at 2d and 14 d, respectively. As for the IC specimens, while the 2-d bond strength is about 18 % higher than the control, the bond strength obtained at 14 d was not significantly different. One aspect to point out about the IC samples is that they showed a larger number of air pockets (e.g., large porosity) at the interface. This might be attributed to the fact that the LWA needed to provide IC is added to the grout material, reducing its paste content per unit volume and slightly changing the rheology by making the material less fluid. It is conjectured that this could have an impact on the bond strength by reducing the contact area. The same effect was also observed at 2d, although the bond strength obtained was slightly larger (perhaps, other variables such as the increased degree of hydration due to the extra moisture have more effect on the bond strength at this age). The tensile strength is always larger than that of the interface failure, confirming that the weakest region is located at the grout-concrete interface, at least up to 14d of hydration (noting that the 2-d IC tensile strength could not be measured).

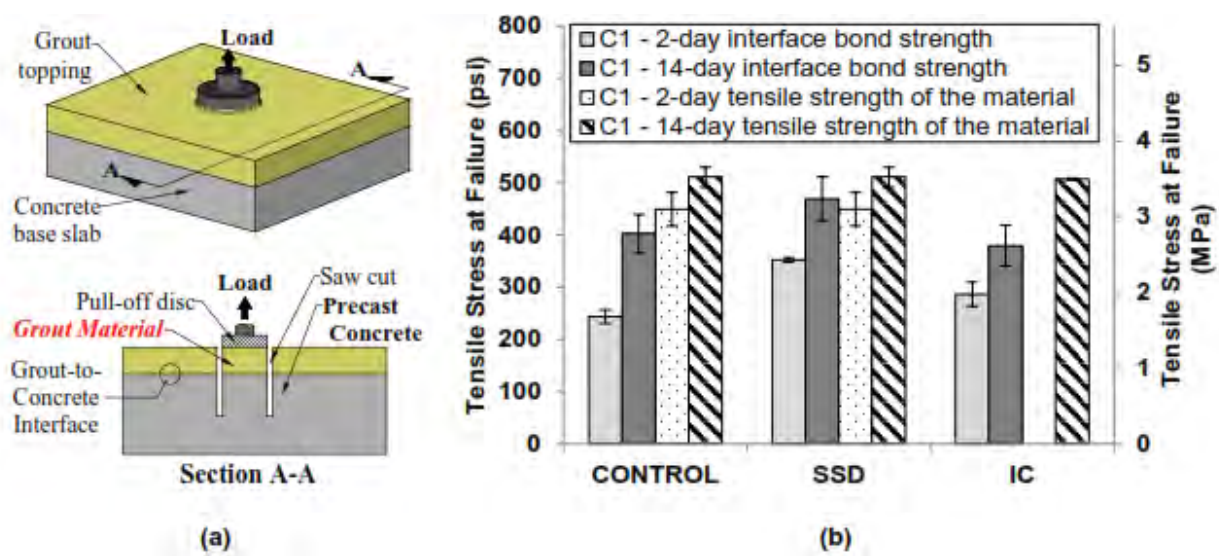


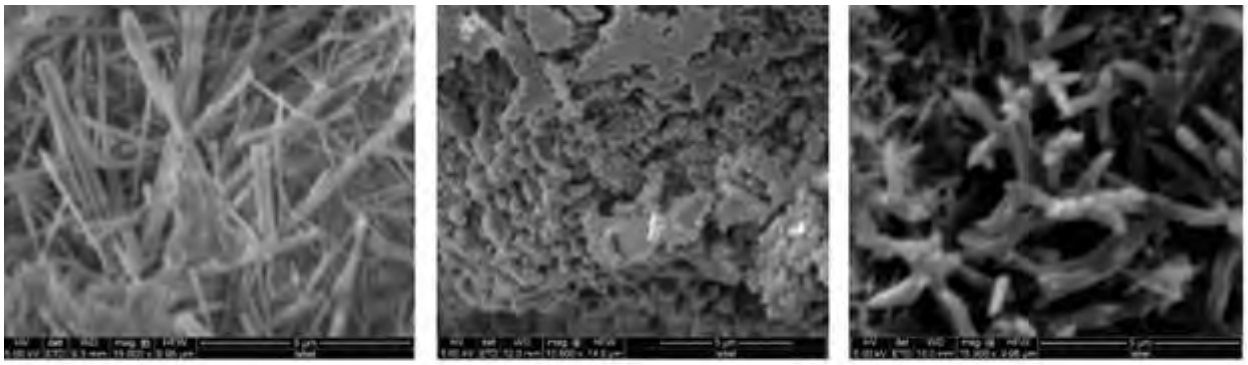
Figure 1. (a) Illustration of the pull-off test on the grout-concrete slab via ASTM C1583, (b) 2-d and 14-d pull-off bond strength for the different interface moisture conditions. (Error bars represent ± one standard deviation from the average of four samples)

DISCUSSION AND CONCLUSION

In an attempt to explain the bond strength results, SEM images were collected from the grout side of the interface in each of the specimens studied: control, SSD, and IC. A clear difference on the type of crystals

formed was observed between the control and the specimens with extra moisture (SSD and IC). While the microstructure on the control specimen was dominated by the presence of large blade-shaped crystals randomly oriented, both SSD and IC specimens showed a microstructure dominated by the presence of denser 'equant' crystals, with a morphology closer to cubical and thick needle shapes (Figure 2b, 2c). The earlier formation of these types of crystal was promoted by the presence of extra moisture at the interface. Eventually, the bladed-shaped crystals of the control specimen will also transform into the denser equant crystals (as observed in the 14-d SEM micrographs, although not shown in this paper).

The presence of extra moisture accelerates this transformation. It is then conjectured that the equant shape of those crystals increases the contact area between the grout and the concrete, compared to that of the blade-shaped crystals (that is, a more interpenetrating contact type).



(a) (b) (c)

Figure 2. 2-d SEM micrographs of the control fractured sample at the grout side of the interface: (a) control, (b) SSD, (c) IC.

In conclusion, the presence of extra moisture (i.e., SSD condition) at the concrete surface changes the type of microstructure present at the interface, increasing the bond strength at the grout-concrete interface.

ACKNOWLEDGMENTS

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ACCELERATED BRIDGE CONSTRUCTION (ABC) TOOLKIT FOR GEORGIA CITY AND COUNTY ROADS

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ABSTRACT

Recently, as a result of increased interest and use of accelerated bridge construction (ABC) in Georgia, the Georgia Department of Transportation (GDOT) funded a research project aimed at introducing ABC design and construction to Georgia cities and counties through the use of a toolkit. The research focuses specifically on short span bridges for span lengths of 40, 60, and 80 ft. The toolkit is not intended to be used for developing a final design, but rather as an informational source that can help decision makers develop an initial design, estimate the material and construction costs, and determine when and where ABC is most beneficial. This paper presents the process used in developing the toolkit and its primary features.

INTRODUCTION

Nearly 25% of the Nation's 600,000 bridges require rehabilitation, repair, or total replacement (1). In cases requiring bridge replacement, Accelerated Bridge Construction (ABC) techniques have the ability to minimize traffic disruptions during bridge renewals and improve the overall quality and durability of bridges. ABC procedures can be applied to emergency replacements as well as to new projects.

The Georgia Department of Transportation (GDOT) is funding a research project aimed at introducing ABC design and construction to Georgia cities and counties through the use of a toolkit. While GDOT already has experience with short (15 to 20 ft.) pre-engineered modular systems and understands the benefits of using prefabricated components, they have less experience with steel and pre-stressed concrete elements at longer spans (40 to 80 ft.). The purpose of this research is to provide an extensive ABC toolkit that local governments could apply in order to assess the suitability of an ABC, develop an initial design, and estimate material and construction costs in cases requiring 40, 60 and 80 feet span lengths.

The first phase of the project involved the creation and completion of a survey which was distributed to several state DOTs. It contained questions regarding the organization's experience with ABC, the level of acceptance of ABC techniques in their state, the number of completed projects in recent years, impediments to the use of ABC techniques, and the ongoing research on ABC topics in the entity's state.

Having evaluated the extent at which ABC is applied at the national level and impediments to its increased use, the majority of the project was devoted to developing a toolkit that contains construction, design, risk analysis, and cost estimate components. The construction guidelines encompass most of the steps used in the construction process, ranging from the foundation excavation to the paving of the deck. It also outlines the construction process of the offsite prefabrication area, transportation of elements, and setting of the prefabricated bridge elements. The design component provides design concepts, user friendly pre-design examples, interactive design flowcharts, and design aides, which will allow city and county engineers to readily follow the extensive procedures involved in ABC bridge design. The risk assessment components of the toolkit focus on evaluating the bridge's ability to convey the design and base floods without causing significant damage to the roadway, bridge, waterbody or adjacent property. The guidelines and interactive flowchart assist the decision maker in determining if a culvert or bridge crossing is most appropriate for the site. They also explain how hydrologic data is used in determining the peak discharges for different design year floods and how discharge is applied to culvert design, a bridge foundation investigation (BFI), and scour analysis. The cost estimate component provides examples of cost comparisons corresponding to both Federal and State requirements. They are included to assist local governments in developing initial cost estimates. The ABC toolkit also provides guidelines to assist local governments and third-party designers in employing GDOT design standards for accelerated-built bridges. Results from the study suggest that continued experience with ABC projects in Georgia will lead to increased contractor acceptance as well as to savings in schedules and costs. This could diminish the initial additional costs.

SURVEYS

The survey was sent out to various agencies to inquire on their experience with ABC. It consisted of questions focusing on gauging the experience of bridge owners: it inquired about both their successes and failures in order to identify what worked and what didn't. Additionally, a more generalized 7-question survey taken from SHRP2 (2) was sent to DOTs of all 50 states to learn about their experiences with ABC. Results from our surveys were obtained from 45 of the 50 states and are summarized using tables and an ABC map (Figure 1) in Appendix B of the GDOT RP 14-10 Final Report (3).

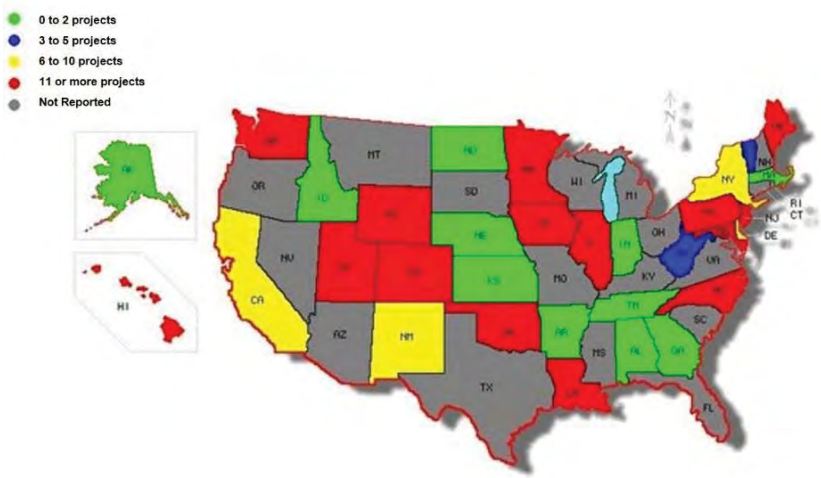

Figure 1. Map on ABC Experiences in the USA showing recently completed ABC construction projects (Survey period: May 2014 to April 2015)

The survey results for owners and contractors identified the following impediments to ABC: 1) elevated costs; 2) lack of experience; 3) constructability issues such as connection details, congestion of rebar around joints, and staging area; 4) hesitation of innovation acceptance; and 5) design-bid-build contract. It also revealed that contractors prefer cast-in-place construction for bridge renewals since large prefabricated elements often diminish profitability (4). Moreover, ABC involves a new technology, and contractors prefer to keep their own employees working instead of subcontracting work to precasters. Possible solutions to overcome impediments to the adoption of ABC are as follows:

- Develop an extensive ABC toolkit that contains all aspects of ABC.
- Introduce the industry to precast technology and demonstrate profitability.
- Use pre-engineered modular systems that can be built with conventional construction equipment, which enable local contractors to bid on rapid replacement project.
- Bundle several bridge projects with similar requirements into a single construction contract, which allow a local contractor to get more benefits with repetition.
- Utilize full-moment connection with ultra-high performance concrete (UHPC), which satisfies the criteria for constructability, structural requirements, and durability in prefabricated modular superstructure systems.

ABC TOOLKIT

Our toolkit has three unique and superior features as compared to existing ABC toolkits:

- **Extensive**

The toolkit not only covers the design and construction components, but also addresses the risk analysis, decision-making tools, and cost estimate components as shown in Table 1. This broad coverage allows the owner, designer, contractors, and decision-makers to make well informed decisions and initiate ABC projects without any additional resources.

- **Convenient**

Enhanced and detailed design examples minimize the need of other design aides such as finite element programs or structural analysis software by providing additional Mathcad design aides to calculate design loadings on superstructures. This information is convenient in creating initial ABC designs.

- **Current**

The toolkit includes current state-of-the-art development of ABC applications through comprehensive literature reviews and latest surveys.

Table 1. Comparison between SHRP2 ABC toolkit (2) and GDOT toolkit (3).

ABC Components	GDOT ABC Toolkit	Existing SHRP2 ABC Toolkit
Decision-Making Tool	Decision-making matrix Decision-making flowchart	N/A
Construction	Construction guidelines Construction flowcharts	Construction concepts Construction specifications (recommendation)
Design	Design concepts Design examples & aides	Design concepts Design examples Design specifications (recommendation)
Risk Analysis	Risk analysis guidelines Interactive flowcharts	N/A
Cost Estimates	Cost estimates guidelines Examples of cost estimates	N/A

The SHRP2 Report (4) and GDOT RP 14-10 Final Report (3) detailed information on the aspects of ABC. The following sections provide an overview of the primary features and attributes for each component of the ABC toolkit.

ABC DECISION-MAKING TOOLS

The ABC Decision-Making Tools is a section devoted to provide guidance on when to use ABC versus conventional bridge construction. If ABC is found to be the most efficient type of construction, then this will also serve as a guide as to which ABC method is deemed most appropriate for a specific project.

A table containing a Decision Making Matrix is developed, an ABC Decision Flowchart (Figure 2), and a Decision Making Scoring Chart with descriptions of items that can be used in conjunction with one another to answer whether to use ABC or conventional methods for bridge construction. The Decision-

Making Matrix may be used to determine how applicable ABC is for a specific project. This matrix is utilized by tallying up the total amount of points next to each section and finding the overall score for a project. After a total score is determined from the Decision Making Matrix, the score is then used to enter the Decision Making Flowchart at the appropriate location. The Decision Making Flowchart is designed to help the user make an intelligible decision on whether ABC or a conventional method is the best decision for the project. Once the correct scoring location is determined the question "Do the overall advantages of ABC negate any additional costs?" is to be answered. These additional costs may include schedule, traffic impacts, funding, road user costs (RUC), etc. This question is to be answered on project-specific basis taking into consideration all engineering components and professional judgement, and also the available project information. This question is a part of the Decision Making Flowchart in order to assist the user of the Decision Making Tools in analyzing and making an intelligent decision on whether ABC is in fact the best form of construction for a project. After answering this question and concluding that ABC is in fact the best method, the part of the Decision Making Flowchart afterwards will help guide the user to the best form of ABC for the project.

The Decision Making Flowchart can incorporate a variety of resources that are pertinent which may include (but are not limited to) "Program Initiatives", like research needs, local resources, input from the public, requests of stakeholders, or structure exhibits. These items should be considered on a project-specifics basis. While the Decision Making Flowchart is designed to lead the user to the best method for ABC, it should be noted that there is room to combine methods listed at the bottom of the flowchart (i.e. PBES, GRS-IBS) if that is what suits the specific project best.

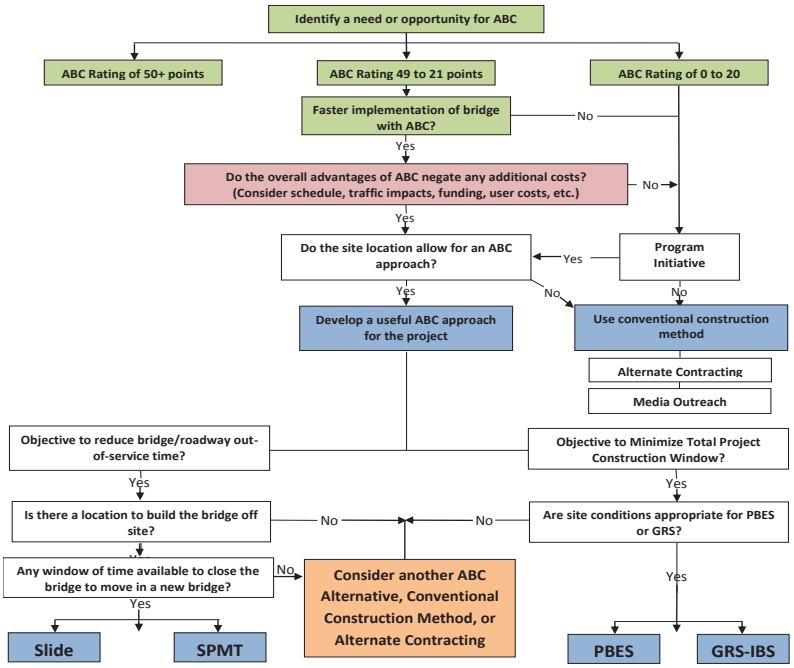

Figure 2. ABC Decision Flowchart Tool (adapted from WisDOT Bridge Manual 2015 (5))

ABC CONSTRUCTION COMPONENT

This component provides an overview of all field-related aspects of ABC construction practices. Early ABC projects focused on specific prefabricated elements, such as bridge decks and/or pier caps. Bridge deck construction using full depth precast concrete deck panels has been in use for over 20 years. In recent years, ABC projects that use Prefabricated Bridge Elements and Systems (PBES) have spread to all bridge components, including substructures and foundations. PBES are structural elements of a bridge that are built off site. Their use in ABC is crucial because it reduces both onsite construction time and the commuter impact time that is generated by conventional construction methods. Combining PBES with the fast-track contracting method (which overlaps the design and construction tasks) can generate a high-performance/fast paced construction project. Figure 3 below shows typical ABC construction procedures for a short time period. The estimated times, on top of Figure 3, are for the construction site only. The fabrication location can be off site or at a country staging area. It, therefore, will not disrupt the traffic flow or any operations on the construction site.

Figure 3. Typical ABC construction procedure.

ABC construction concepts are summarized in Table 2. Detail application examples and technology information for all bridge components can be found in the GDOT final report (3) and SHRP2 Report (4).

Table 2. ABC construction concepts (4)

Item	Description
ABC Construction Concepts	Prefabricated Spread Footings / Precast Pile Cap Footings Modular Block Systems Geosynthetic Reinforced Soil Integrated Bridge System (GRS/IBS) Expanded Polystyrene Geofoam (EPS) Prefabricated Modular Abutments or End-Bents System Prefabricated Superstructure Elements Materials for Prefabricated Bridge Elements
ABC Construction Technologies	Above-deck driven carrier systems / Launched temporary truss bridge Wheeled carriers or SPMTs / Launching and lateral sliding Jacking and mining
Application Examples of ABC Construction Technologies	Deck/Superstructure Replacement / Substructure Replacement Existing Bridges Replacement Staging / Full Closure and New Construction

ABC DESIGN COMPONENT

Design Concept

This component of the toolkit summarizes current developments of ABC design concepts that can be used for future ABC projects through the state of Georgia. Table 3 below presents several current state-of-the-art ABC design concepts and considerations presented in greater detail in GDOT 2015a. It should be noted that ABC projects use innovative designs that are also compatible with innovative construction techniques due to the interconnection between design and construction. ABC design strategies for rapid renewal of bridges can be expressed as follows (4):

- **Minimize Weight**

This concept improves the load rating of existing foundations and piers, and can simplify the transportation and erection of bridge components.

- **Simplify Design**

To achieve this goal, it is recommended to reduce the number of certain elements, such as girders, field splices, and bracing systems.

- **Simplify Building Process**

Fewer workers and fresh-concrete operations on site are desirable. Additionally, geometry needs to be simple.

Table 3. ABC Design Concepts and Detail Considerations

Item	Design Concepts	Descriptions
Modular Superstructure Systems	Decked Steel Stringer System Composite Steel Tube Girder System Precast Concrete Deck Bulb Tee/ Double Tee Pre-Topped Trapezoidal Concrete Tub Beams Full-Depth Precast Concrete Deck Systems UHPC Superstructures Connections Between Modules	Two beam steel sections with slab Trapezoidal steel box girders suitable Easily adopted by experienced contractors Texas DOT U beams used CIP joints replaced by match cast joints Compressive strengths of 18,000psi to 30,000psi Crucial to the speed of construction
Modular Substructure Systems	Integral and Semi-Integral Abutments Precast Abutments and Wingwalls Precast Complete Piers Hybrid Drilled Shaft/Micropile Foundation Systems Steel or FRP Jacket System for Existing Column	Jointless Construction Precast components as light as practicable Attached to the foundation by grouted splice sleeve connectors Composed of conventional drilled shaft and clusters of micropiles For retrofit and accelerated construction

Figure 4. ABC Superstructures Interactive Design Flowcharts

Design Examples

This section provides user friendly pre-design examples and interactive design flowcharts as shown in Figure 4. Both steel and concrete girder design examples were developed for 40, 60, 80 ft. span lengths, and modified to allow for easy understanding. The base design examples were taken from the SHRP 2 document "Innovative Bridge Designs for Rapid Renewal" (2). Modifications were made to the original design document by using GDOT standard criteria (6) for highway bridges, information obtained from a design example created by the Federal Highway Association, and the latest AASHTO LRFD Bridge Design Specifications, 6th Ed. (7). All design examples in this project were created using Mathcad, which allows city or county engineers throughout the state to readily follow the extensive procedures involved in ABC bridge design. Simplicity is stressed throughout the examples.

Design Aides

An analysis of the superstructure can be performed using structural modeling software or computational aides to calculate the design moments, shears, and reactions. Whereas the existing SHRP2 design examples use finite element analyses, in order to ensure that the toolkit can be employed by users without access to finite element software, it provides Mathcad-based examples to evaluate the maximum design loadings using the qBridge software (8).

RISK ANALYSIS

This portion of the toolkit addresses the need to convey surface water and/or storm water runoff to minimize damage to the roadway, bridge, and other property. The toolkit is largely informative and provides the user with basic information relating to: storm return periods (20 year, 50 year, etc.); the use of statistical analysis in determining the probability of occurrence and how this relates to risk; guidelines and processes for selecting either a culvert or bridge crossing for the project site; the process for using hydrologic data to determine the peak flow (discharge); and how peak flow is used for sizing culverts or in BFI and scour analysis.

The Role of Risk in Culvert and Bridge Design

Since rainfall events are governed by chance, historical rainfall information and statistical analyses are used to estimate the magnitude of different storm events over different return periods. Risk is a measure of the probability of occurrence multiplied by the cost associated with repairs/replacement caused by the event. Because the return period is inversely related to probability that the storm will occur that year, there is a direct relationship between the storm period and risk. While larger return periods have lower probabilities of occurrence, the tradeoff is that they also have higher construction costs. For example, using a return period at 20 years (probability of equaling or exceed the storm is 1/20 or 5%) might result in a project with a low initial construction cost but with frequent repair or replacement expenditures. Conversely, using a large return period of 200 years (probability of equaling or exceed the storm is 1/200 or 0.5%) can result in an overly designed project with an excessive construction cost. Since the selection return period requires careful consideration of several factors (potential damage to highway and property as a result of flooding, potential hazards and inconveniences to the public, and project costs), GDOT specifies the return periods for both culvert and bridge projects as follows (9):

- Culverts for state routes and interstate highways shall be designed using a 50-year flood frequency.
- Bridges for state routes and interstates shall be sized so that a 50-year flood is conveyed only through the bridge opening and the 100-year flood is conveyed through the bridge opening and over the roadway.

Selecting between a Culvert and Bridge Crossing

Culverts are closed conduits that convey surface water or storm water runoff from one side of a road to the other side. They play a key role in preserving the road base by preventing water from overtopping the road surface and by keeping the sub-base dry by draining water from ditches along the road. Whereas bridges use the bridge deck, superstructure (beams, girders), and substructure (abutments, piers) to

support vehicle loads, culverts rely on the structural properties of the conduits and the embankment material covering them to support these loads. In cases in which the toolkit user needs to decide between using a culvert or bridge crossing, the following guidelines should be applied (3):

Table 4. Guidelines for Choosing Between a Culvert and a Bridge Crossing

Conditions for a Culvert	Conditions for a Bridge Crossing
Area draining to the crossing does not exceed 20 miles^2 (12,800 acres).	Area draining to the crossing exceeds 20 miles^2 (12,800 acres).
Surface water is limited or not present.	Cases in which the surface water canal is navigable.
The water area is well defined (i.e. easily be routed through the culvert).	The water area at the crossing is undefined.
Cases in which large debris will not pass below the roadway	If the crossing point is located near an area where flow back up behind the culvert could flood residential areas.
Cases in which any flow backup will not flood adjacent areas	If high debris loads (gravel, trees, logs) passing below the roadway are likely.
	Cases in which the hill catchments is steep.

Determining Peak Flow and Its Application

As shown in Figure 5 below, the process for computing the peak flow starts with a computation of the watershed area using either the United States Geological Survey (USGS) StreamStats application or other approaches such as Google Earth, counting squares, a planimeter, etc. If the watershed area is less than 0.3 miles^2 (200 acres) and urbanized (having impervious areas of 10% or greater), the toolkit directs the user to apply the Rational Method to compute peak runoff. In cases in which the watershed is rural, or urban but with an area between 0.3 miles^2 (200 acres) and 9,000 miles^2 (5,760,000 acres) the peak runoff is computed using USGS regression equations. These equations are provided for five hydrologic regions within Georgia with watershed as shown in GDOT RP 14-10 Final Report (3). Having computed the peak runoff, the toolkit shows the user how to apply this value to either size a culvert (single or multi-barrel) or as a parameter in the BFI and scour analysis for a bridge crossing.

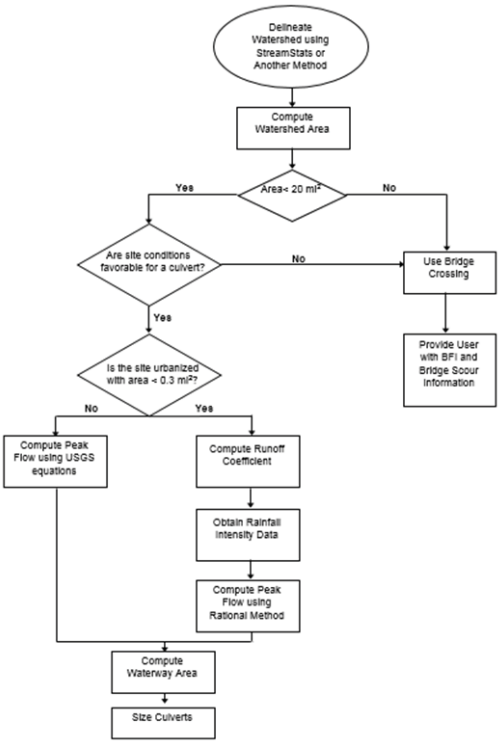

Figure 5. Risk Analysis Flowchart

CONCEPTUAL COST ESTIMATES

The primary concern for ABC construction cost estimation is the higher cost associated with the projects. This has resulted in higher bid prices due to the complexity of the project and the severity of the time constraints imposed on the contractors. Some agencies that have expressed concerns over cost, indicate that they do not see a need to spend extra funds to minimize impacts to the public. In fact, these agencies are already spending additional funds for this purpose. Most agencies employ phased (staged) construction in order to keep traffic flowing through a work zone. It is well known that this construction method is more expensive than construction with a full road closure. The contractors are required to work in a small work zone with adjacent traffic that impedes the work in progress. The approach can increase the cost of the construction. ABC allows owners to take reductions in traffic impacts to the next level by providing even better customer service. In some cases, it may be preferable to close the roadway, establish a detour and build the bridge quickly using ABC. The additional costs of this process may very well be offset by the elimination of phased construction costs. As more ABC projects are built, the costs are trending downward due to construction familiarity with the process which results in lower risks. Lower risk equates for lower costs in the project. For this toolkit, examples are studied to provide a better understanding on cost benefit comparison for use of ABC vs. Conventional construction. Life cycle cost analysis is a process for evaluating the total economic worth of a usable project segment by analyzing

initial costs and discounted future costs, such as maintenance, user costs, reconstruction, rehabilitation, restoration, and resurfacing costs over the life of the project segment. Also, higher quality reduces the need for maintenance and extends the lifespan of the structure, which will lead to a reduced life cycle cost for prefabricated structures. As cost accounting options for the toolkit, two approaches were considered: Unit Cost and Cost-based Estimating. Unit Cost approach is an accounting method that combines the cost for time, equipment, manpower, materials, general and project-specific overhead, contingency and profit. Cost Based (Bottom-Up Estimating) is a method of accounting that takes into consideration production rates, equipment needs, and manpower for each construction operation.

SUMMARY AND CONCLUDING REMARKS

Accelerated Bridge Construction (ABC) techniques are highly effective in minimizing traffic disruptions during bridge renewals, promoting traffic and worker safety, and improving the quality and durability of bridges. Despite lower long-term costs and the life-cycle cost savings, the higher initial cost of employing ABC technologies has prevented widespread and sustained implementation. The survey results from this investigation also revealed that local contractors prefer conventional cast-in-place construction for bridge renewals as modular construction diminish profitability. Furthermore, designers hesitate to be involved in a new ABC technology due to the level of risk associated with ABC projects. Comprehensive studies for ABC practices in the nation, however, have found that accumulated experience and repeated use of the ABC practices could lead to contractor acceptance as well as to savings in construction costs and time.

The primary objective of this study was to develop and deliver an extensive ABC toolkit for accelerated selection and construction of bridges in place using prefabricated modular systems with 40-, 60-, and 80-ft span lengths for local governments in Georgia. The ABC toolkit was developed to assist decision makers in 1) determining if ABC is an appropriate technology for the solution of their bridge-related problem, and 2) developing an initial design with an estimate of material and construction costs. As such, the toolkit provides local governments and third party consultants with an extensive, convenient, and current set of guidelines for implementing various ABC applications. The toolkit is not intended to be used for developing final designs and construction tasks, but rather as an informational source that can assist decision makers and owners in developing an initial design, estimate costs, and determine when and where ABC is most beneficial. The study concludes that, with more successful and repeated implementations, ABC options will become more economical and widespread.

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NOTATION

Acronym	Definition	Acronym	Definition
ABC	Accelerated Bridge Construction	PBES	Prefabricated Bridge Element System
BFI	Bridge Foundation Investigation	RUC	Road User Cost
CIP	Cured-In-Place	SHRP2	Second Strategic Highway Research Program
DOT	Department of Transportation	SPMT	Self-Propelled Modular Transporter
EPS	Expanded Polystyrene Geofoam	UHPC	Ultra High-Performance Concrete
FRP	Fiber Reinforced Polymer	USGS	United States Geological Survey
GDOT	Georgia Department of Transportation		
GRS/IBS	Geosynthetic Reinforced Soil / Integrated Bridge System		

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SHRP2 ABC DETAILS LEAD TO A SUCCESSFUL BRIDGE REPLACEMENT IN MAINE

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INTRODUCTION

This project involved the use of Accelerated Bridge Construction (ABC) techniques to replace an aging single span concrete rigid frame highway overpass structure in less than 30 days. This was one of seven pilot projects selected to receive SHRP2 R04 funding for implementation of the ABC Toolkit.

EXISTING CONDITIONS

The Kittery Overpass Bridge is a single span structure that carries the U.S. Route 1 Bypass over State Route 236. The existing bridge was originally constructed in 1941, and had become structurally deficient due to extensive deterioration to the concrete rigid frame.

Existing Bridge

Traffic congestion at the bridge can become quite heavy, especially during the summer months, due to the bridge's proximity to popular tourist destinations on Maine's southern seacoast. The average annual daily traffic (AADT) on US Route 1 is 7,740 vehicles per day, while State Route 236 has an AADT of 20,000 vehicles per day.

PRELIMINARY DESIGN

This design team evaluated both conventional and accelerated construction methods during the preliminary design phase of the project. A long-term closure of the U.S. Route 1 bypass was undesirable, and phased bridge construction was greatly complicated by the concrete rigid frame bridge system and the shallow bedrock a the site. Based on input from the local community and a desire to minimize impacts to the public, the MaineDOT elected to utilize precast bridge elements and systems, in combination with

Accelerated Bridge Construction methods to replace the bridge during a single 30-day roadway closure period.

FINAL DESIGN

The replacement structure is a 60' single span bridge consisting of PCI Northeast Extreme Tee (NEXT-D) Beams with Carbon Fiber Composite Cable (CFCC) prestressing, which represents one of the first applications of this material in the United States. The NEXT beams were supported by full-height precast concrete abutment stems and cast-in-place concrete footings founded on bedrock. The project also utilized precast concrete modular gravity (PCMG) wingwalls, and the completed bridge contained a total of 166 separate precast concrete elements.

The presence of shallow bedrock at the site required the development of several details to mitigate the time required for rock removal. The SHRP2 ABC Toolkit was utilized in the development of many of the precast components, in combination with cast-in-place footings due to the irregular bedrock surface. In addition, alternate footing connection details were developed to mitigate project risk, and these were successfully utilized during the closure period. Also, due to high traffic volumes on the roadway beneath the structure, limited closure of this roadway was allowed. Contract language and financial incentive provisions were developed by the design team to encourage a rapid bridge demolition that could be completed in less than 24 hours.

CONSTRUCTION

The project was advertised for construction in January 2014 with bids received from six General Contractors. The successful bid was submitted by Wyman & Simpson, Inc. of Richmond, Maine for a total bid price of $2.8M. Fabrication of the precast bridge elements occurred between April and June, with the 30-day roadway closure period scheduled to begin in late July.

11-Hour Rapid Bridge Demolition

The U.S. Route 1 Bypass and State Route 236 were closed to traffic on July 23, 2014 to accommodate the rapid bridge demolition. Shaw Brothers Construction, the earthwork subcontractor completed the demolition and reopened State Route 236 to traffic after only 11 hours, earning a $65,000 incentive for early completion of this work item.

Cast-In-Place Footing Construction

Following the rapid bridge demolition, the earthwork subcontractor utilized the same tracked excavators; fitted with hydraulic rams to mechanically remove 500 cubic yards of bedrock, which was required to accommodate the new abutments and wingwalls.

An unreinforced subfooting was then placed over the excavated rock in order to provide a level working surface to form and construct the cast-in-place reinforced abutment footings. Pre-assembled forms and reinforcing steel cages were then placed on the prepared subfooting. The abutment excavation, subfooting, and footing construction tasks were completed in eight days.

Following a two-day wait period for initial footing concrete strength gain, vertical precast abutment stems were set into place on top of the footings with temporary braces installed to resist lateral wind and construction loading.

Full-Height Precast Abutments

The precast abutment stems were detailed with corrugated metal pipe (CMP) block-outs both to reduce the weight of the pieces and to facilitate a full moment connection to the footing. Precast horizontal cap beams were then placed on top of the abutment stems to help distribute the abutment loads and to serve as the bearing seat for the precast NEXT beam superstructure. Once the grouted connections achieved the required strength, the abutments and wingwalls were backfilled up to the proposed roadway subgrade

Erection of Precast NEXT-D Beams

At Day 21 of the roadway closure period, the precast NEXT beam superstructure system was erected using a single 275-ton All-Terrain hydraulic crane. Route 236 below the bridge was closed for a 12-hour period to accommodate the beam erection and forming of the longitudinal closure joints between the adjacent NEXT beams. High-early strength

concrete was used in the longitudinal closure joints, and the superstructure was completed with the installation of a high performance spray applied membrane system and asphalt wearing surface. The approach roadway was also reconstructed and the U.S. Route 1 Bypass was reopened to traffic after 29 days.

LESSONS LEARNED

A few valuable lessons were learned, which could be applied to similar ABC projects in the future, including:

- Shallow bedrock is a schedule risk, but can be mitigated on ABC projects through additional subsurface investigations and the use of cast-in-place subfootings/footings where appropriate.

- The use of circular column-type reinforcement connections between the abutment footing and stems is an effective detail that reduces costs and increases construction tolerances compared to splice sleeve connectors.

- Separate Incentive/Disincentive provisions for each affected roadway can be used successfully to minimize impacts to traffic.

CONCLUSION

This ABC project was well received by the public and helped to validate and advance the use of the SHRP2 ABC Toolkit. The project utilized innovative carbon fiber prestressing; included an 11-hour rapid bridge demolition; more than 500 cubic yards of rock removal; is comprised of 166 separate precast bridge elements; and successfully replaced an aging structurally deficient bridge in a 29-day roadway closure period.

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John P. Auger
Resident Engineer

Maine Department of Transportation

STRENGTH, DURABILITY, AND CONSTRUCTABILITY OF GROUTED COUPLERS FOR INTEGRAL ABUTMENTS IN ABC PROJECTS

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ABSTRACT

The benefits of integral abutment bridges have rarely been tied to the Accelerated Bridge Construction movement for various reasons. There is currently little documentation for integral abutments being used for ABC, so this study aims to develop, construct, and test integral abutment details that utilize grouted couplers to splice the pile cap to the integral diaphragm. Two details were investigated in this study, an integral abutment using grouted rebar couplers, and an alternative detail named the pile coupler. The grouted rebar coupler's performance was comparable to that of a traditional cast-in-place abutment, and while the benefits of the pile coupler detail are clear, the detail is still a work in progress.

INTRODUCTION

The benefits of integral abutment bridges have been realized and used by engineers and contractors as early as the 1920's. The main benefit of an integral abutment bridge is that there is no expansion joint present on the superstructure. This feature leads to a bridge that is fast and inexpensive to construct because of simplified abutment geometry and having only one row of abutment piling. Furthermore, by removing exposed beam bearings and expansion joints, two critical areas that require routine maintenance, the lifetime cost is lower for integral abutment bridges.

Accelerated bridge construction has made an impact on the bridge world, rethinking the typical construction procedures and techniques by moving a significant portion, if not all of the component construction time, off of the critical path. Since ABC is relatively new, there is still largely unexplored territory in terms of design details, construction details, and more importantly the documentation of performance for the many different types of bridges and details being built using ABC. The strength, durability, and constructability of integral abutments using grouted couplers is unknown in large part because they has they have seen very limited use. For this reason a research project was created by the

Bridge Engineering Center (BEC) in order to document the design and testing process of ABC details specific to integral abutment bridges.

METHODOLOGY

The strength, durability, and constructability of integral abutments using ABC techniques is untested and largely unknown. In order to create and expand the knowledge base in this area, three full-scale specimens were constructed and tested in the structures laboratory at Iowa State University. In order to get a better understanding of the constructability of the details investigated, the process of fabricating and constructing each specimen was well documented by the research team. To evaluate durability and strength, each specimen was load tested to simulate live load, thermal expansion and thermal contraction. The loads were designed to test the connection between the pile cap and the integral diaphragm for crack widths present at service loads, and also for ultimate capacity. Durability is investigated by measuring crack widths between the joint of the pile cap and integral diaphragm in order to understand the threat level for water, chlorides and debris to penetrate the joint. The specimen were also subjected to ultimate loading, in an attempt to discover or predict what load would fail the design. In order to establish a baseline for the ABC details being tested, one cast-in-place specimen was built and tested as a control specimen.

SPECIMEN DESIGN

To design the three specimens, namely the cast-in-place, the grouted rebar coupler, and the pile coupler specimens, the Iowa DOT standard integral abutment detail was used and modified to fit the needs of the ABC connections and those of the structures laboratory. The cast-in-place specimen used for this research project is shown in Figs. 1 and 2, where the “8g1” rebars provide tension and compression reinforcement to the connection between the pile cap and the integral diaphragm. The 8g1 bars also bend into the deck in order to provide continuity for the negative moment region in the deck above the girder.

The grouted coupler specimen, see Fig. 3, is similar to the cast-in-place specimen, except with a precast joint between the pile cap and the integral diaphragm, where the “8g1” rebar have been spliced using grouted couplers. A grout bed was added to the precast joint to alleviate any imperfections in the concrete’s finish and insure an even distribution of loading across the joint. Additional rebar was added to the ABC details to add strength for resisting shear and flexure while the specimens in the lab, or precast elements in the field, are being transported. The specimen constructed in the lab was limited to 8 feet in width, due to crane capacity, which is similar to the maximum width of a precast element that could be transported on the highway. For the laboratory specimen there were 17 rebars and grouted couplers that

required alignment between the diaphragm and the pile cap. While this number may not be challenging for experienced contractors and prefabricators, a full width bridge constructed utilizing slide-in technology, attempting to use grouted rebar couplers for the integral abutment connection would add considerable tolerance and constructability concerns to the project.

The pile coupler detail was developed at the BEC in an attempt to alleviate the constructability concerns encountered with the grouted rebar coupler detail, specifically for slide-in bridge construction. The pile coupler detail aims at reducing the number of grouted connections that need to be made for the integral abutment connection, facilitating a full slide-in bridge. For example, if there were 180 spliced rebar connections to be made on a 45 foot wide bridge, the pile coupler detail would only require 24 grouted connections. The pile coupler detail is shown below in figures 4 and 5, where a CMP is used to create a large void in the center of the abutment, and an HP section of steel and a grout mix is used to fill the void, connecting the pile cap to the integral diaphragm. The pile coupler works similar to a grouted rebar coupler, however the entire HP section of steel is suspended and contained in the integral diaphragm, such that transportation and or sliding of the superstructure over the pile cap is allowed without obstruction from protruding reinforcement. Once the superstructure is in place over the pile cap, the HP sections are lowered into the CMP void in the pile cap so that the precast joint bisects the HP section, and the void is grouted. Two ports exist on the outside of the abutment to access the CMP void containing the pile; the first is a 3” PVC pipe used to grout the CMP closed; the second is a 1” PVC pipe which doubles as a way of lowering the cable attached to the HP section, and as a vent for grouting the void. The 1” PVC pipe is tilted slightly upwards so that the entire CMP will be filled with grout before grout exits the vent.

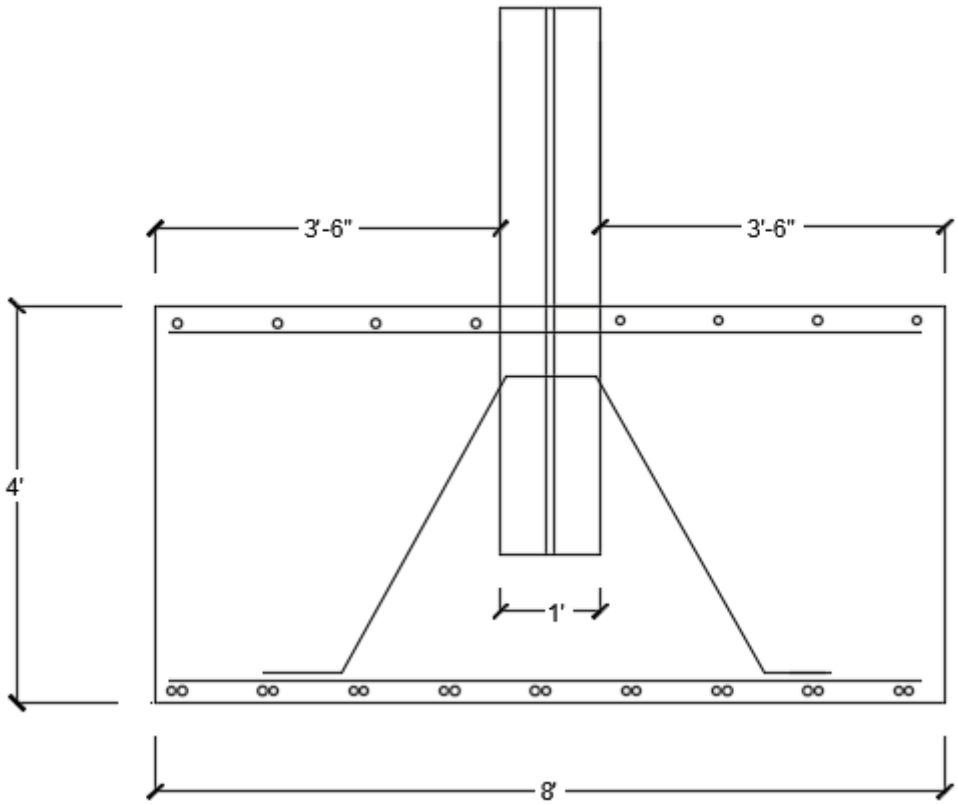

Figure 1 - Plan view cast-in-place specimen

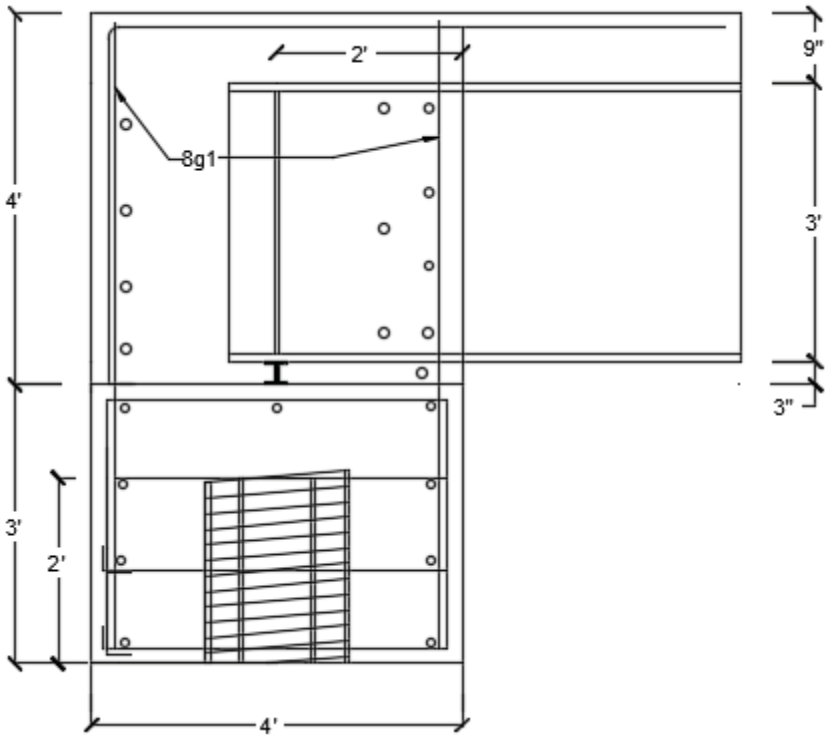

Figure 2 - Section view of Cast-in-place detail

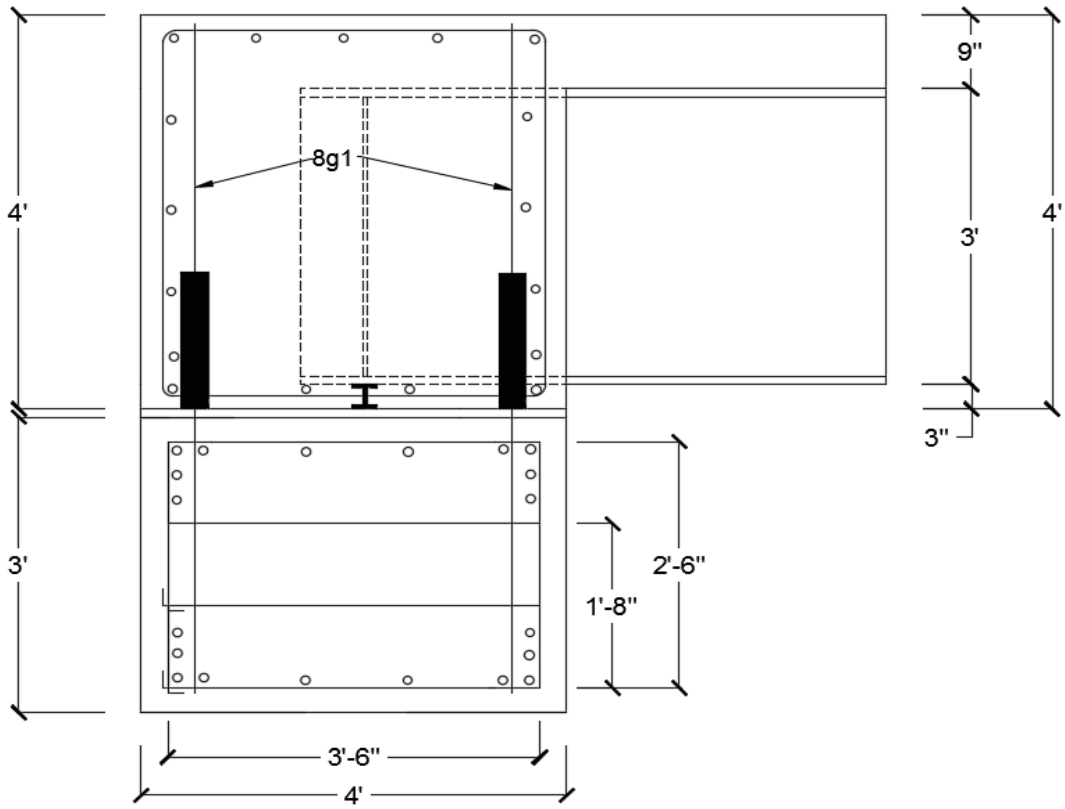

Figure 3 - Section view of grouted coupler detail

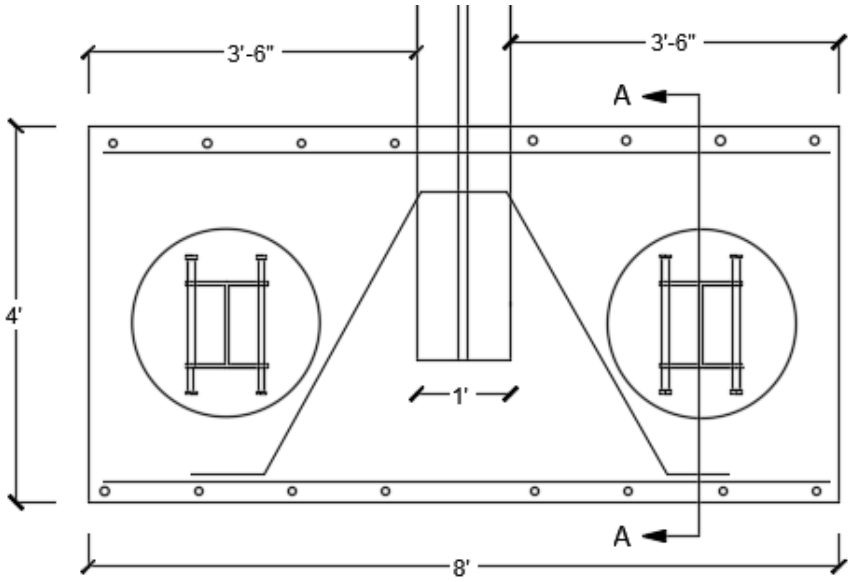

Figure 4 - Plan view pile coupler specimen

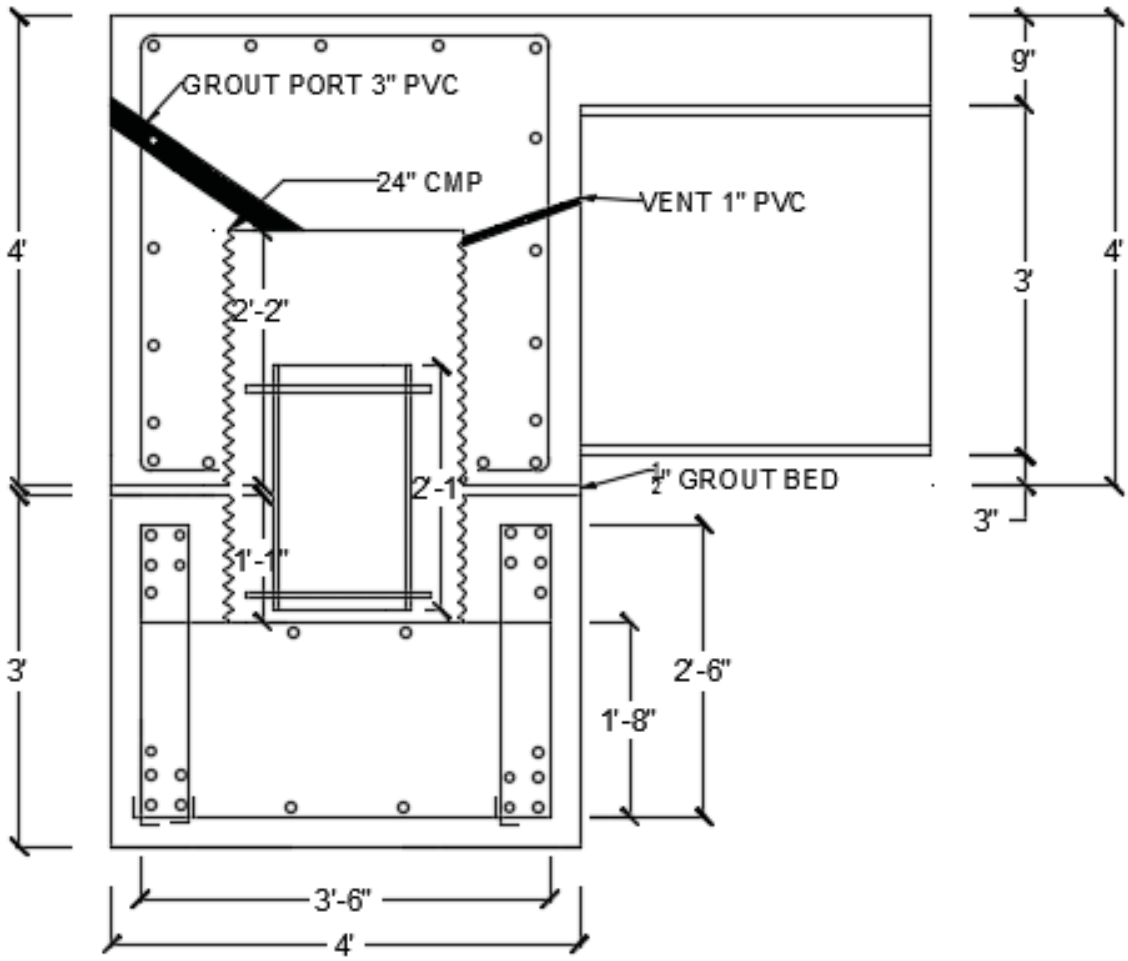

Figure 5 - Section view A of the pile coupler detail

SPECIMEN CONSTRUCTION

Construction of the cast-in-place specimen was completed much like would be done in the field on an actual bridge; the pile cap rebar was tied, formwork was erected and concrete was poured and broom finished. Next, the integral diaphragm rebar was tied, formwork was erected, and the concrete was poured and finished. The completed specimen, along with the reaction blocks used to fix the specimen to the testing floor is shown below in Fig. 6.

Figure 6 - Cast-in-place specimen

To construct the pile cap for the grouted coupler specimen, the same procedure was followed as the castin-place specimen. However, to insure alignment of the 17 rebar and grouted couplers connecting the pile cap to the diaphragm, a template was made from plywood which marked the exact locations of the rebar on the pile cap, see Fig.7. Holes were then drilled into the template at the marked rebar locations and form plugs were installed to tighten and hold the grouted couplers in position for construction of the integral diaphragm.

Figure 7 - Grouted rebar coupler template

Rebar was then tied around the grouted couplers on the template and the integral diaphragm was cast.

With the integral diaphragm and pile cap complete, the surfaces of the pile cap and integral diaphragm were wetted to the SSD condition, see Fig. 8, in preparation for the grout bed. Once the diaphragm was sitting on the ½" shims on the pile cap, formwork was installed around the perimeter of the joint and pumping of the grout bed commenced. The following day the grouted couplers were grouted; 15 of the 17 couplers were grouted successfully, the other two couplers appeared to have grout from the grout bed that leaked into the grouted coupler, caused by either an unsatisfactory seal or excess pressure from pumping the bed.

Figure 8 - Integral diaphragm placement

To construct the pile coupler specimen, the pile cap was cast with two, 24 in. diameter x 12 in. CMP lengths to create a void which receives the HP section from the superstructure. Next, the integral diaphragm rebar was erected and two, 24 in. diameter x 24 in. CMP lengths were placed in the center of the diaphragm on either side of the girder. Each CMP in the diaphragm section has a metal lid fabricated with 4 rebar guides penetrating down into the CMP, which act as a guide during the lowering of the HP section, see Fig. 9. The lid was also outfitted with a U-bolt anchored near the center such that a wire cable could be attached to the HP section, run through the U-bolt, and the run out the 1" vent to facilitate lowering of the HP section. The HP section had holes drilled through the flanges and threaded rods with nuts were installed to increase connectivity with the grout inside of the CMP. Overall, the pile coupler

specimen was simple to construct, lowering of the pile and grouting of the voids was straightforward and the system as a whole seemed efficient in terms of construction.

Figure 9 - CMP with lid, rebar guides and U bolt

Figure 10 - HP section with threaded rods

INSTRUMENTATION

Strain gauges, displacement transducers and a load cell were used in the laboratory to capture and record information during load testing. For all three specimens, displacement transducers were installed on the tension face of the abutment to measure the width of the crack that developed at the cold joint of the cast-in-place specimen and precast joints of the other two specimen. Strain gauges, shown as red circles in Fig. 11, were placed on various rebar in the cast-in-place and grouted rebar coupler specimens to capture or aid in predicting the load at which the steel would yield within the concrete. For the pile coupler specimen, the pile was instrumented with strain gauges to capture data from the testing.

TEST RESULTS

The cast-in-place specimen and grouted coupler specimen were tested to the full ability of the structures laboratory, without seeing any signs of distress in the abutment other than the crack that opened at the cold/precast joint between the pile cap and the integral diaphragm.

Engineers design the connection between pile cap and the integral diaphragm to be stronger than the plastic moment capacity of the H-piles used in the foundation. To evaluate the strength and durability of the designs, a 4 foot spacing for foundation piles was assumed, and given the 8 foot wide test specimen, an adequate design must be stronger than two abutment piles while maintaining a reasonable crack opening. To convey this information, crack width vs. applied bending moment is illustrated in Fig. 11 below, where the horizontal lines are the plastic moment capacity for given pairs of foundation piling.

Figure 12 illustrates similar data, this graph shows results from the horizontal load that was applied to the deck section, which simulates thermal contraction.

The pile coupler specimen showed significant signs of distress during vertical loading of the specimen.

Large cracks started to form early on in the loading, which was not the case with the other two specimen.

The ultimate capacity of the detail was reached during the vertical loading, signified by significant rotation of the integral diaphragm and a crack between the diaphragm and the pile cap measuring approximately 1.75 inches. It was at this point that the testing was over, as the specimen could no longer resist the applied load, and the resulting abutment is shown in Fig. 13.

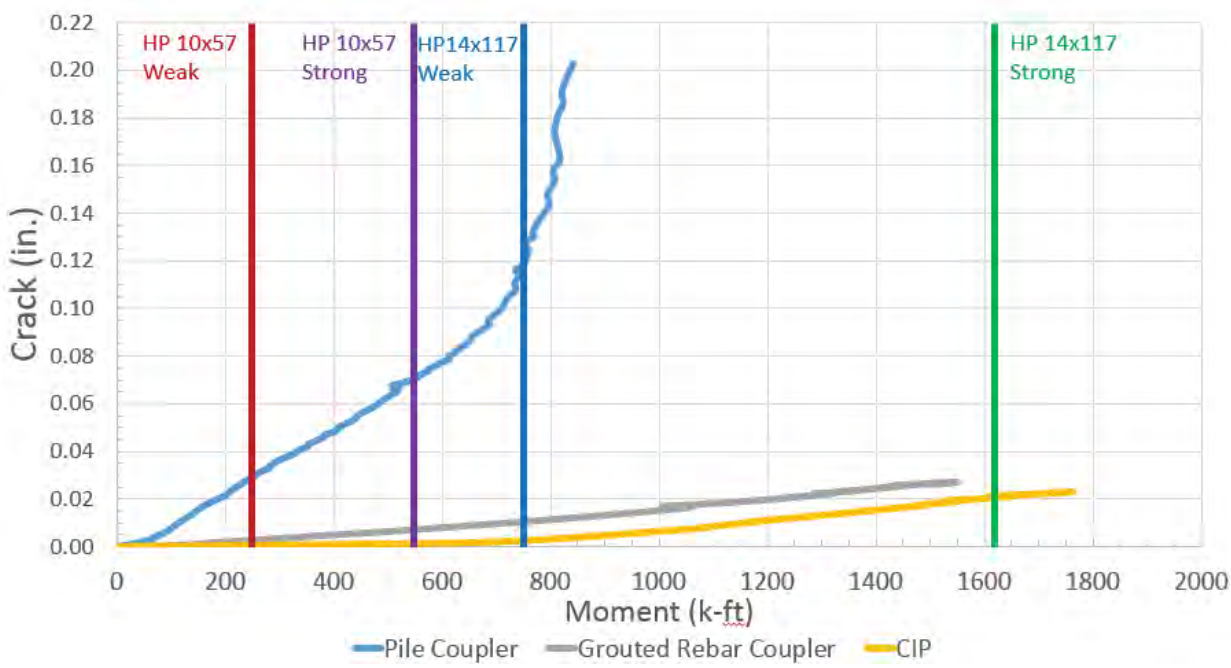

Figure 11 – Abutment piles Mp (two piles), vertical loading vs crack width

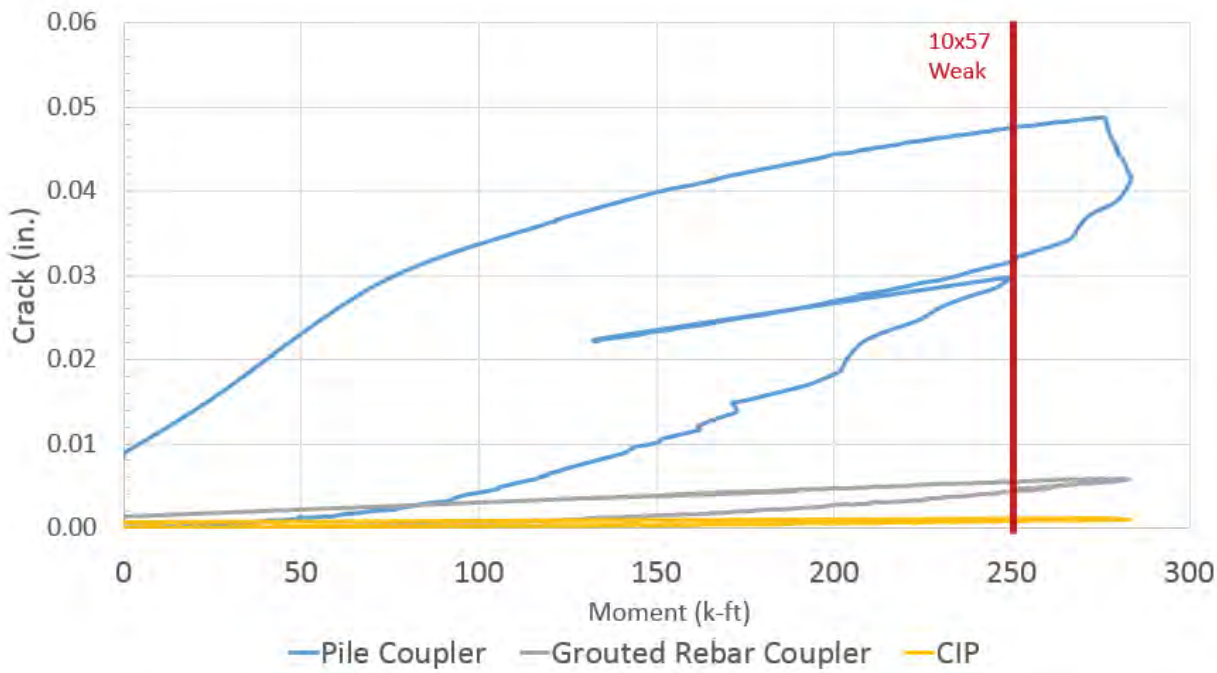

Figure 12 – Abutment piles Mp (two piles), horizontal loading vs crack width

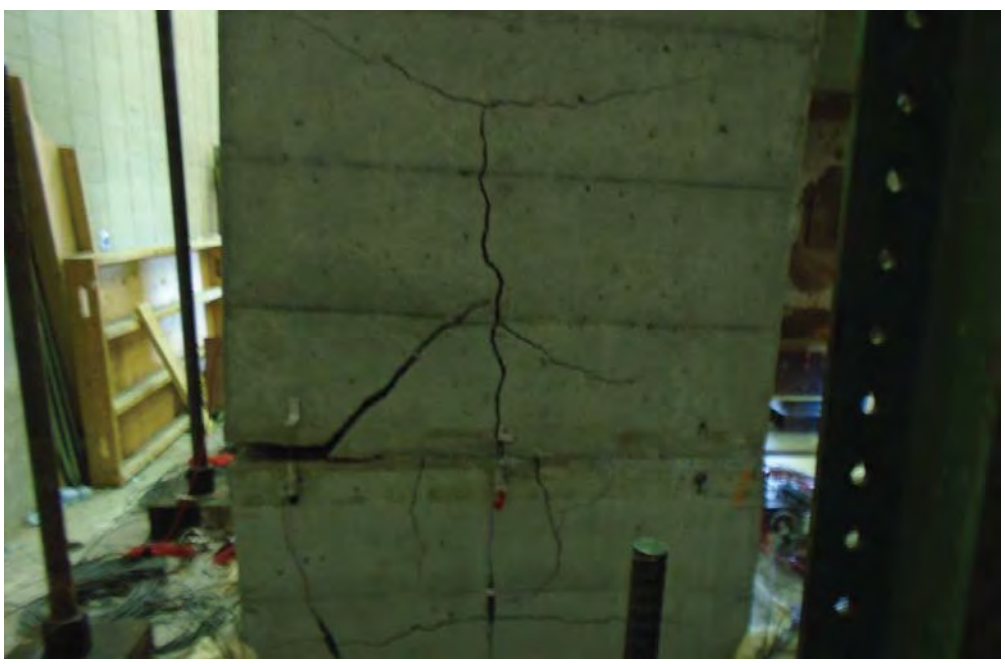

Figure 13 - Pile coupler cracking

CONCLUSIONS

The ABC integral abutment detail using grouted rebar couplers exhibited comparable performance with the cast-in-place detail. The maximum crack width for the grouted rebar coupler detail was approximately

0.035 inches, which is slightly larger than that of cast-in-place specimen crack width of 0.02 inches measured at the same load. The overall strength of the grouted coupler specimen was 16% less than the cast-in-place specimen, which is a comparison based on the predicted load required to yield the rebar in the two specimens. The reduction in strength of the grouted coupler design is most likely do to two things, the two un-grouted couplers, and the smaller effective section depth, resulting from the grouted couplers requiring additional concrete cover. In terms of overall strength, the goal of the integral abutment design is to develop the plastic moment capacity in the abutment piles. Based on this criterion the rebar coupled integral abutment joint is adequate for even the strongest of foundation piles. While the grouted rebar coupler detail had a sound structural performance, constructability may be an issue, unless the size of the precast element is limited to have a realistic number of grouted coupler connections. Further complications in constructability arise should this detail be attempted in slide-in-bridge construction.

In theory, and on paper, the benefits of a detail like the pile coupler were obvious. However, the performance, in terms of strength and serviceability of the detail in laboratory testing was less than ideal, though it did alleviate the constructability issues encountered with the rebar coupler design. Moving forward, there are possible changes being investigated by the research team to improve the strength and durability of the pile coupler detail. Which include using two pile couplers side by side, while looking at a section view of the abutment, to improve the moment arm, and distribution of reinforcing steel.

G-12: ADVANCEMENT OF PREFABRICATED BRIDGE ELEMENTS

ABC CONSTRUCTION WITH AN INNOVATIVE AND COST-EFFECTIVE FULL DEPTH PRECAST DECK SOLUTION

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ABSTRACT

Sponsored by FHWA Every Day Count program, The Bayou Lafourche Bridge is currently under construction, applying the AccelBridge technology. AccelBridge is a cost effective full depth precast deck system which utilizes bridge girders to compress the deck without need for deck post-tensioning. Comparing with current full depth precast deck technologies, AccelBridge offers advantages in simplifying construction, reducing cost, and improving durability.

INTRODUCTION

Full depth precast decks are one of the most widely used Accelerated Bridge Construction (ABC) methods for bridge superstructure construction, due to their ability to be applied to conventional girder structures. Current full depth precast deck systems ensure the long-term integrity of deck panel joints either through the use of internal deck post-tensioning to supply pre-compression across the joint interfaces or through the use of cast-in-place Ultra High Performance Concrete (UHPC) joint closure pours. However, both methods have tended to complicate construction and substantially increase costs. AccelBridge is an innovative full depth precast deck system that improves constructability and reduces cost by utilizing girder jacking methods to apply the needed and beneficial deck compression without deck post-tensioning. Its offers significant savings in material/labor costs when compared with conventionally post-tensioned deck systems or those deck systems employing UHPC joints. In addition, AccelBridge achieves enhanced durability by putting the entire deck in longitudinal compression, in contrast to a UHPC-based system, and by eliminating the corrosion potential of conventional deck post-tensioning. AccelBridge also simplifies construction and reduces traffic downtime when compared with compared with current full depth precast deck construction methods by eliminating cast-in-place panel joints and post-tensioning duct coupling operations.

FHWA has sponsored Louisiana Department of Transportation and Development (LADOTD)'s implementation of AccelBridge via the Innovative Bridge Research and Development (IBRD) program. With a goal of developing a set of design and construction standards to apply this practical ABC

technology for the DOTD's most widely used structure type, LADOTD selected the Bayou Lafourche Bridge, a seven span, prestressed girder supported bridge. The construction contracted is awarded in 2014 and construction starts at April of 2015.

The bid price of Bayou LaFourche indicated that AccelBridge costs much less than other full depth precast deck methods. To accurately gauge the cost, LADODT specified an individual pay item for jointing the deck with AccelBridge jacking method (separated from the precasting of deck panels). The bid price of jointing the deck with AccelBridge jacking method ranges from $ 1.21 to $1.36 per sq. ft of deck for the lowest three bidders. In comparison, the average cost is $7.50 per sq. ft. for post-tensioning system and $15.00 per sq. ft. for UHPC jointing method.

ACCELBRIDGE SYSTEM

AccelBridge System is a set of patented technologies utilizing a full-depth precast deck. It retains the durability advantages of post-tensioned deck panels with compression stress across the transverse joints, but contains no internal post-tensioning or post-tensioning ducts. Accel_NP system is the technology used in Bayou Lafourche bridge. The AccelBridge detailed presented in this paper focuses on the NP system. Readers can refer to www.accelbridge.com, for information on other AccelBridge systems.

In the Accel_NP System, the precast deck is jacked against the girders, creating tension in the girders and compression in the bridge deck. The typical construction steps for the Accel_NP System are:

1. Erect the precast deck panels on the girders.
2. Make the last panel on one end of the bridge composite with the girder while all the remaining deck panels are free to move in the longitudinal direction.
3. Place a jacking assembly at the other end of the bridge, and jack the deck panels against the steel girder.

Figure 1 shows schematics of the deck panel jacking to provide the needed longitudinal compression. More detail information can be found from reference He (1).

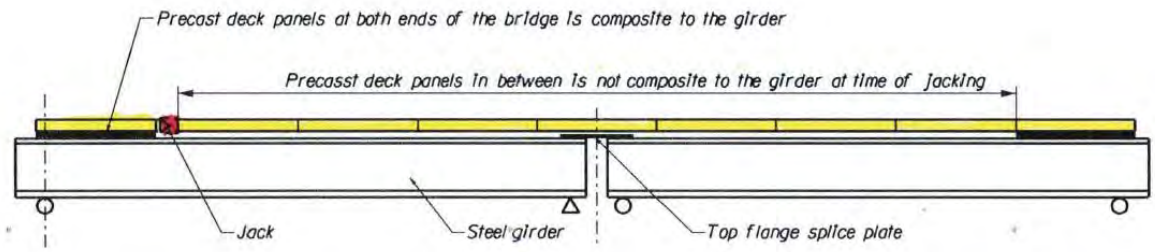

Figure 1. Accel_NP System Working Principles

At the time of jacking, only deck panels at both ends are fixed to the girder while all other panels in between can move relatively to the girder. The jacking force then results in compression in the deck and tension in the girder. After all panel to girder shear connector is grouted, the deck compression will be permanently locked in. The simple method allows the deck in compression without any post-tensioning.

By eliminating deck internal post-tensioning, the Accel_NP System offers the following advantages over current practice:

- No embedded PT ducts in the precast deck panels.
- No duct coupling and duct blockage potential at the deck panel joints.
- No tendon stressing and grouting operations.
- No corrosion concerns for PT tendons.

The concept of the Accel_NP System can be applied to most types of girder bridges, concrete or steel, I girder or box girder, single span or multiple spans. The Accel_NP System can also be used for deck replacement on an existing bridge.

Besides saving all PT materials and labor, this jacking method also typically benefit the girder. Since the jacking force is at deck level, and offset from girder center, the jacking also introduces a negative moment in the girder, which reduces the girder moment demand at midspan, where typically controls the girder section. This cost saving potential is particularly attractive for projects on deck replacement of existing girders. Accelbridge deck jacking can potential increases the structure load rating and eliminates the need for strengthening of existing girders.

PROJECT OVERVIEW

Bayou Lafourche bridge carries US-80 over the Bayou Lafourche, LA. This is a complete replacement. The proposed bridge is 560 feet long and uses a 41 feet wide deck with one travel lanes in each direction,

consisting of 7 spans at 80 ft each. The full depth precast deck will be supported by six lines of AASHTO type III girders. The elevation and transverse section of the bridge is shown in Figures 2 and 3.

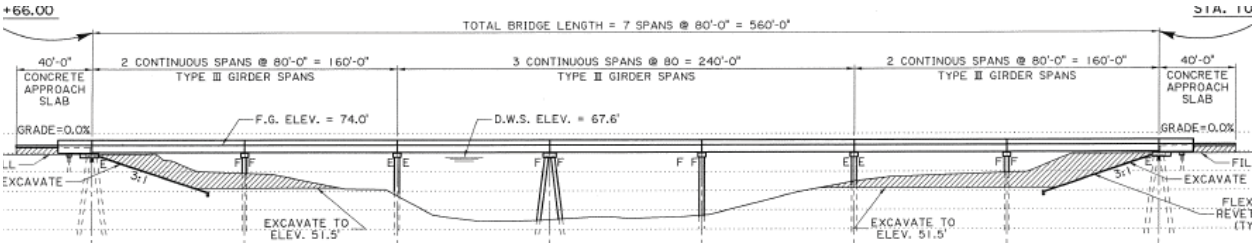

Figure 2. Elevation of Bayou Lafourche Bridge

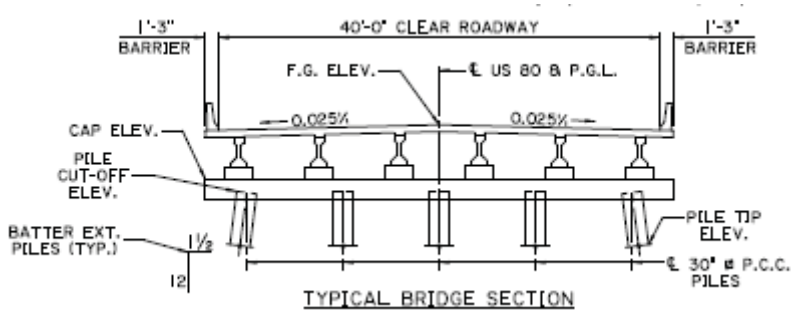

Figure 3. Cross section of Bayou Lafourche Bridge

Figure 4 shows the overall view of the superstructure system. The precast panels are very similar to typical current practice of full depth precast decks, with the following exceptions:

1. There is not deck post-tensioning.
2. Deck joints are match cast joint with epoxy.
3. A jacking gap is provided to apply the deck compression.
4. Girder top flanges are connected at the pier locations

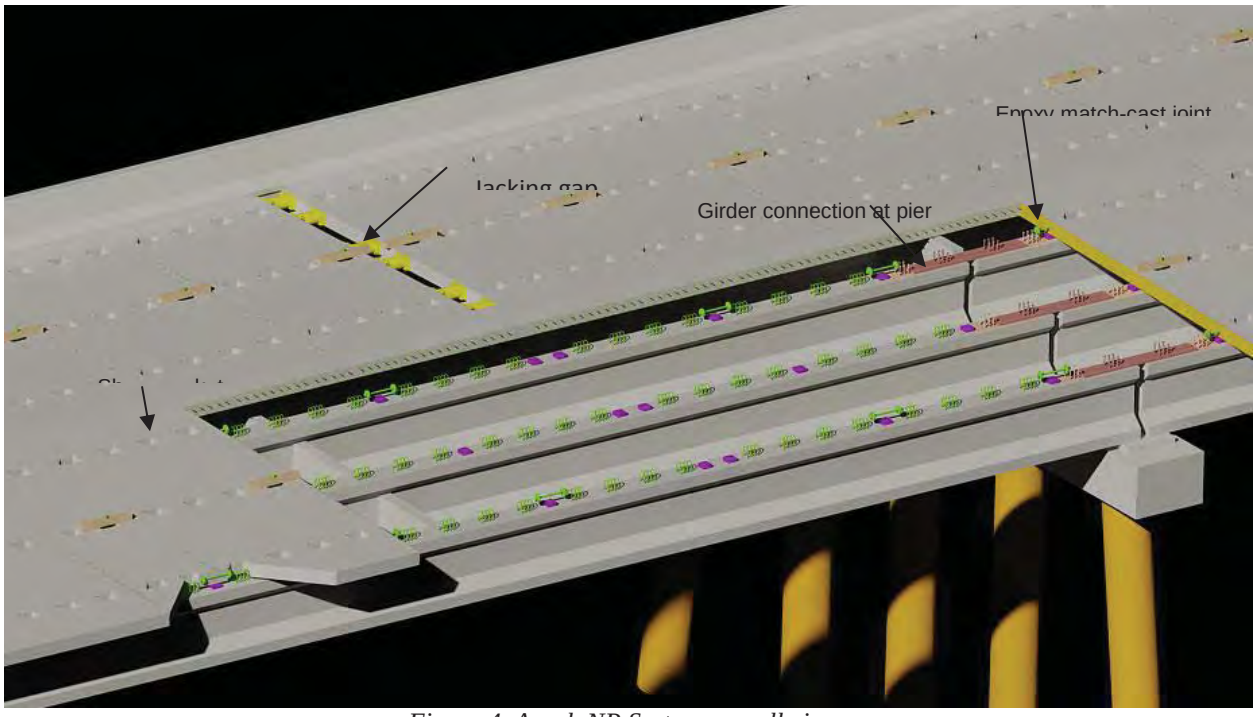

Figure 4. Accel_NP System overall view

Figure 5 shows the jacking layout for Bayou Lafourche. Jacks used are typical hydraulic jacks with locknuts. A total of 12 jacks are used to apply 2000 kips to the deck, which equivalent to about 460 psi in deck compression stress.

Figure 5. Jacking details

Precast deck section

The bridge deck slopes to both sides with a crown at the center. The current design allows two options for the precast deck:

1. To cast the deck in two halves, with a field longitudinal closure in the middle. This will allow precastors to cast all panels flat.
2. To cast entire deck section as one unit.

Girder connection at pier

AccelBridge required the girders within the same jacking unit are continuous to resisting the girder tension force resulted from deck jacking. The girder connection at pier is constructed by two steps.

a) A field welded plate connection to make girder top flanges to be continuous, before deck jacking occurs.
b) Concrete diaphragm is poured after the deck erection is complete. At this time, girders are momently connected at pier location.

It is worth noting that only step a) is required for Accelbridge. The moment connection of girders at pier is optional. The full moment connection detail at piers in Bayou Lafourche is used for the purpose of following LADOTD's standard practice.

The field welded girder connection at pier can be found from Figure 4. The field weld connection is part of the means for construction only; it is used to apply compression between deck panels. After the completion of panel installation and grouting of shear pockets, the compression force in the deck joint is locked in and the weld connection is no longer needed.

Deck details

Other than the method of applying longitudinal deck compression, AccelBridge system utilizes proven details from typical full depth precast deck system or segmental bridges. The key details are: Deck to girder connection and joints between deck panels.

Deck to girder connection details in Bayou Lafourche Bridge is the same as the shear pocket details in typical full depth precast deck projects. The connection between deck and girder are accomplished by rebars projecting from girders, grouted haunch and shear pockets in the precast deck.

Some of shear pockets adjacent to the joint also houses the erection bolt, which is used to provide the needed 40 psi stress across panel joint to compress the epoxy. The erection bolts uses the typical high strength bolt and can be torque to the desired tension by wrench. This is a simplification from the erection PT bars in segmental bridge erection.

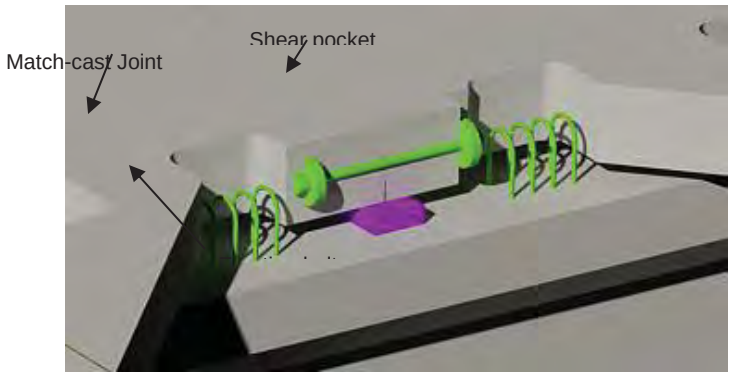

Figure 6. Deck panel joint details

AccelBridge can work with both conventional cast-in-place (CIP) joint and match-cast joint with epoxy. Bayou Lafourche Bridge employs match-cast joint, which is preferred due to better durability and easier field installation.

Match-cast deck panels can be produced in a manner similar to that of precast segmental, but greatly simplified in terms of geometry control and form work setups. One of the difficulties with the match-cast in segmental bridge is geometry control, which has been a major concern of precastors. However, deck panels in AccelBridge can be cast flat without consideration of the actual profile of the bridge. Since the depth of precast deck is small (only about one-tenth of that for a typical segmental box designs), the joint opening due to deck vertical curvature is small and can be absorbed by the epoxy grout layer; eliminating the need to consider the vertical profile in deck panel casting results in significant construction simplification and cost savings.

The long line match casting method seems to be most suitable for full depth precast deck. Panels are poured in two sets, at a “checkerboard” pattern to achieve match-cast. After the first set of panels was poured, the forms were removed from the edge that would butt up to the next set of panels. The second set of panes was poured directly against the surface (with keyway) of the first set of panels, so that the pieces would fit perfectly. The joint between the two panels had a release agent applied to it so when the concrete cured they could be separated. MODOT pioneered the match-cast method for full depth precast deck in the Nemo Bridge project. The Nemo bridge project successfully demonstrated that the match cast method is very straightforward, and was perfectly executed by a contractor with no previous experience in match-cast technology. Details of this project can be found from reference MODOT (2).

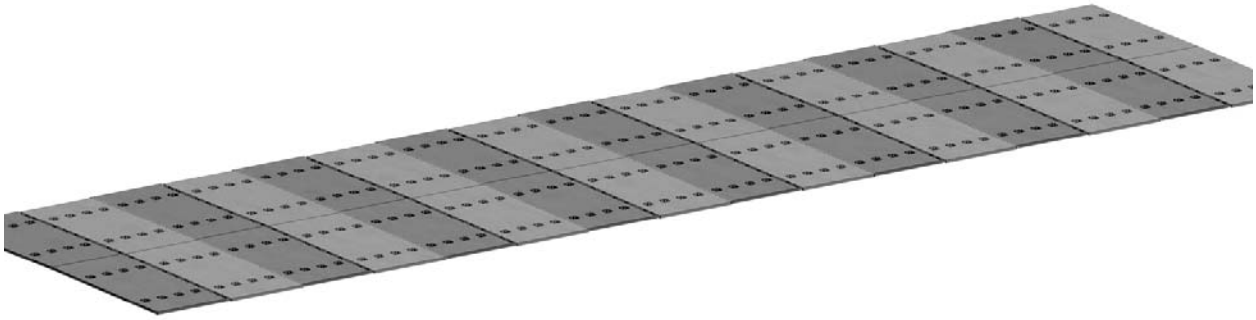

Figure 7. Long line match-casting with checker-board pattern

CONCLUSIONS

AccelBridge System is a set of patented and patent-pending technologies utilizing a full-depth precast deck. It retains the durability advantages of post-tensioned deck panels with compression stress across the transverse joints, but contains no internal post-tensioning or post-tensioning ducts. By eliminating internal post-tensioning, the main construction difficulties for precast deck are addressed. Therefore, AccelBridge greatly improves the constructability, material efficiency, cost and speed of the current precast deck system. In addition, AccelBridge system can achieve better durability than deck systems with internal post-tensioning by reducing or eliminating the corrosion potential of deck post-tensioning.

The AccelBridge systems build upon proven technologies that when used in combination, create a cost-effective full depth precast deck solution, with advantages in:

- Improving durability - It retains the durability advantages of post-tensioned deck panels with compression stress across the transverse joints, but contains no internal post-tensioning ducts. Therefore, the concern of PT corrosion in typical full depth precast deck is completely eliminated.
- Improving durability – Historically, epoxy / match-cast joint demonstrated superior durability, comparing to cast in place joints typically used in full depth precast deck.
- Construability – Eliminating the deck internal post-tensioning not only reduces the amount of work in precast yard or in the field, it also greatly reduces the risk of common PT installation problems such as duct misalignment, imperfect duct coupling, leaking of grout.
- Cost - By eliminating internal post-tensioning, or the expensive UHPC joint, AccelBridge has significant lower cost (estimated at $ 8 – 15 per sq ft), comparing with current full depth precast deck method.

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2. *MODOT, "Deck Replacement with Precast Concrete Segments" http://library.modot.mo.gov/RDT/reports/Ri04027/or06003.pdf*

ACKNOWLEDGEMENT

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CONSTRUCTION OF THE SECOND GENERATION OF PRECAST CONCRETE DECK SYSTEM NUDECK

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ABSTRACT

This paper presents the first implementation of the 2nd generation of NUDECK system to the Kearney East Bypass project in Kearney, NE. The project has twin bridges: south bound bridge constructed using cast-in-place concrete deck; and north bound bridge constructed using the new full-depth full-width precast concrete deck system. Each bridge is a two-span continuous bridge that is 41 ft 8 in. wide and 332 ft long, and has five precast/prestressed concrete girders (NU1800) at 8 ft 6 in. spacing per span. The successful experience of fabrication and erection of ten NU1800 and twenty eight NUDECK panels along with post-tensioning and grouting operations are summarized.

BACKGROUND

Full-depth precast concrete deck systems have several advantages over cast-in-place (CIP) concrete decks in bridge construction, such as improved construction quality, reduced construction time and impact on traveling public, possible weight reduction, and lower bridge life-cycle cost. The quality of precast concrete decks is superior to field-cast concrete decks because production occurs in a controlled plant environment eliminating the variability in quality due to weather conditions, casting operations and curing techniques. Moreover, the risk of shrinkage cracking is significantly reduced, if not eliminated, in precast concrete decks due to the use of high performance concrete, two-way prestressing, and delaying the connection to the stiffer girders that restrain deck deformation.

Precast concrete deck systems can be designed as composite or non-composite with the supporting girders. While a non-composite deck is less expensive than a composite one, the composite design results in smaller girders because of the superior structural performance. The composite deck systems are therefore more economical and commonly used. Traditional full-depth composite precast concrete deck systems use either continuous open channels along the girder lines or open discrete pockets at a 2 ft maximum spacing to accommodate the shear connectors of the supporting girders. These channels or pockets are then grouted and the deck surface is protected by an overlay similar to CIP deck systems,

which increases construction duration and cost. In addition, transverse joints in the traditional precast concrete deck systems are either conventionally reinforced or post-tensioned by threading strands through embedded ducts along the bridge length. These joints and/or ducts have to be protected using special grout. These operations complicate the fabrication and erection processes and, consequently, reduce the attractiveness of precast concrete deck systems as an accelerated, economical, and durable alternative to CIP deck systems.

The objective of this project was to develop and implement a precast concrete deck system that addresses the shortfalls of the traditional systems. Table 1 presents a side-by-side comparison of the proposed system and traditional full-depth full width precast concrete deck systems with respect to four criteria. The use of longer precast deck panel reduces the number of panels, transverse joints, and cast-in-place operations. Increasing the spacing between shear connectors simplifies the fabrications of both the precast girder and deck; and minimizes conflicts in matching connectors with pockets during erection. Also, using covered pockets with limited penetration results in a more durable deck surface that does not require an overlay and reduces life-cycle cost. Placing post-tensioning strands over each girder line prior to deck placement simplifies the post-tensioning operation and eliminates the need for threading strands and grouting ducts. After all, the goal of the owner is to achieve a deck life expectancy that matches the bridge design life, which is 75 years according to AASHTO LRFD.

Table 1: Comparing the proposed and traditional precast concrete deck systems.

Criteria	Traditional Precast Concrete Deck Systems	Proposed System
Panel Length	8 ft or less	12 ft
Shear Connectors	Either continuous connectors or discrete at a maximum spacing of 2 ft	Discrete connectors at 4 ft spacing and possibly 6 ft
Panel Penetrations	Continuous open channel or open pockets requiring deck overlay	Covered pockets with grouting ports that do not need an overlay
Longitudinal Reinforcement	Conventional reinforcement or post-tensioned strands threaded through embedded ducts	Post-tensioned strands pre-placed in the haunch underneath deck panels

The recent developments in precast concrete deck systems were proposed based on Nebraska Department of Roads (NDOR) experience in the construction of the Skyline Bridge, Omaha, NE in 2004 using the 1st generation of precast concrete deck system called NUDECK. Although this system was highly successful and received two PCI Awards of Excellence, several refinements were made to further improve its constructability and cost-effectiveness, which led to the development of the 2nd generation NUDECK. Table 2 shows a side-by-side comparison between the 1st and 2nd generations of the NUDECK system.

Several analytical and experimental investigations were conducted at the University of Nebraska-Lincoln (UNL) to evaluate the structural performance and constructability of the new system. These include the fabrication of a full-scale deck panel to demonstrate panel production and handling operations, fabrication of a 60 ft long precast/prestressed concrete I-girder (NU900) with embedded shear connectors, conducting pull-out and push-off tests to evaluate the interface shear capacity of these connections, and constructing a 60 ft long composite girder with five 12 ft long deck panels to conduct post-tensioning and grouting operations. A self-consolidating concrete (SCC) mixture was developed specifically for this application to allow easy flow of concrete through 4 in. diameter grouting holes and complete filling of the gap between the girder and deck soffit as well as shear pockets. A composite specimen was tested to evaluate the flexural capacity and stiffness of the new system as well as the interface shear capacity of the deck-to-girder connection. Testing results indicated that the measured capacity of the proposed system exceeds the predicted capacity of a fully composite system. For more information, refer to the final report titled "Implementation of Precast Concrete Deck System NUDECK (2nd Generation)"

Table 2: Comparing 1st and 2nd generations of NUDECK systems

Item	1st Generation NUDECK	2nd Generation NUDECK
Panel Length	8 ft	12 ft
Panel Thickness	6 in. + 2 in. type K cement overlay	8 in. without overlay
Panel-Girder Connection	12 in. wide continuous open channel at each girder line filled with conventional	16 x 8 x 5-1/2 in. HSS covered individual pockets at 4 ft spacing filled
Shear Connectors	1.25 in. diameter studs at 6 in. spacing	Two 1.25 in. diameter coil rods at 4 ft spacing
Transverse Pre-tension	4 - 0.5 in. (top) and 4-0.5 in. (bottom) diameter strands at 24 in. spacing	6 -0.6 in. (top) and 6-0.5 in. (bottom) diameter strands at 2 ft spacing
Longitudinal Post-tension	0.6 in. diameter strands threaded through the open channel at deck mid-thickness	0.6 in. diameter strands laid down at the haunch area below deck panels

IMPLEMENTATION

The 2nd generation of precast concrete deck system NUDECK was implemented in the construction of Kearney East Bypass over the US-30 and Union Pacific Rail Road in Kearney, NE. Construction was completed in September of 2015. The project consists of twin bridges: south bound bridge constructed using conventional CIP deck; and north bound bridge constructed using the new precast concrete deck system. This is to compare the construction and long-term performance of the two bridges when subjected to the same environment. Each bridge has 41 ft 8 in. wide and 332 ft long two-span continuous deck

supported by ten prestressed concrete girders (NU1800) at 8 ft 6 in. spacing. The precast concrete deck is 8 in. thick and consists of 28 panels. Figures 1 and 2 show the precast/prestressed concrete girders and deck panels produced by Coreslab Structure, Inc. Plattmouth, NE for the north bound bridge. Figure 1 also shows the shear connectors (2 – 1.25 in. diameter A193 Grade B7 coil rods) at 4 ft spacing along the girder and the custom made post-tensioning strand deviators (for 12 -0.6 in. diameter strands) located at 4 ft from the abutment end of each girder. Shear connectors were greased to allow cranking them up to achieve the required embedment in the shear pockets despite the girder camber. Figure 2 shows the three covered pockets located over each girder line at 4 ft spacing and the 4 in. diameter grouting holes located at the center of each pocket. It also shows the anchor blocks embedded in the end panel for post-tensioning and the projected epoxy-coated bars required for cast-in-place rail construction.

Figure 1: Precast/prestressed concrete NU1800 girders for the new precast concrete deck system

Figure 2: Precast/prestressed concrete deck panels of the new system.

After girders were erected and shim shots were taken, similar to conventional bridge construction, galvanized steel bent plates were welded to the metal tabs embedded in the girder top flange as shown in Figure 3 to support precast deck panels and achieve the required deck profile. This deck support system also works as side forms for grouting the haunch area. Compressible material was attached to the top of bent plates and caulking material was installed along the edges of the top flange to seal the haunch area and prevent leakage of SCC during grouting. Figure 3 also shows the 12-0.6 in. diameter strands installed longitudinally over the top flange for each girder line through the deviators and the 10 #8 Grade 60 bars installed for live load continuity over the intermediate pier (negative moment zone).

Figure 3: Deck support system, post-tensioning strands and continuity reinforcement.

Figure 4 shows the lifting of a full-depth full-width panel (50,000 lb) using 8 lifting inserts embedded in the shear pockets to minimize deck surface penetration. It also shows the installation of the first panel over the pier location. Remaining typical panels were then installed in both direction until end panels were installed at the abutment locations as shown in Figure 4 (installation duration ≈ 30 mins per panel).

Figure 4: Handling and installation of precast concrete deck panels.

Figure 5 shows one of the transverse joints between deck panels (2 – 3 in. wide) and the placing of 6 in. slump conventionally vibrated concrete at the joint using a bucket and shovel. This operation required about 7 yd^3 and was completed in approximately 3 hours. Transverse joints were covered immediately

with curing compound then later with wet burlap. Once the joint concrete achieved at least 3,500 psi compressive strength, post-tensioning was conducted using a mono-strand jack as shown in Figure 6 to apply 500 psi stress. All the 60 strands were tensioned initially to 2 kips, then tensioned finally to 43.9 kips (0.75 f_{pu}) in a symmetrical manner to minimize eccentricity effects on the deck. Measured elongations of all strands were with ±5% of the predicted elongation (25.5 in.) and the total elastic shortening of the deck was 0.5 in. (as predicted)

Figure 5: Placing concrete for transverse joints.

Figure 6: Post-tensioning strands and anchor block.

The grouting of the haunch, shear pockets, and anchor blocks were conducting using highly flowable (slump flow = 28 – 30 in.) and high strength (f'_c = 9 ksi) SCC. SCC was pumped using 3 in. diameter hose from one location (at the bridge pier) to the 4 in. diameter grouting holes on the deck surface using

the apparatus shown in Figure 7 (hopper, bucket, and inverted cone). SCC was placed from one grouting hole and flowed in both directions until overflowed from the adjacent holes, which were covered with sand-filled buckets before moving to the next grouting hole (24 ft away). This process ensured complete filling of haunch and pockets and took 1 hr. per girder line.

Figure 7: Post-tensioning strands and anchor block.

Figure 8 shows the deck surface after grouting and curing of SCC, while Figure 9 shows the final deck surface after placing the rail and grinding the deck surface up to ½ in. The final deck surface will not have

an overlay and will be monitored for any signs of deterioration in the next few years. It should be noted that few transverse cracks were observed at the transverse joints before deck post-tensioning. These cracks were closed after post-tensioning was applied and the final deck surface does not have any cracking. Also, there is no clear discoloration in the final deck surface between precast concrete and CIP concrete used in grouting the transverse joints and shear pockets.

Figure 8: Deck surface after the grouting operation

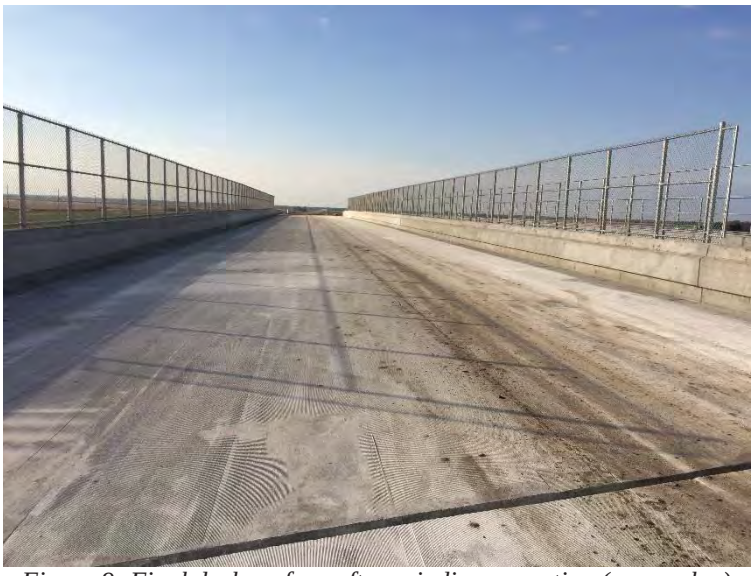

Figure 9: Final deck surface after grinding operation (no overlay)

ACKNOWLEDGMENT

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BRIDGE IN A BOX

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What do companies like LL Bean, Amazon, Dell and Wal-Mart all have in common? They are really successful at customization and transportation logistics. How can these supply chain management techniques be used to take Accelerated Bridge Construction to the next level? There is a developing bridge technology that will revolutionize how small and medium span bridges are designed, manufactured, delivered, and installed in days instead of weeks and months.

- This paper will introduce a modularized, prefabricated bridge system that utilizes advanced materials that can be assembled quickly by semi-skilled labor without heavy equipment.
- The standard components of this system are: arch tubes, decking, and headwalls.
- The system can be shipped as a kit that includes the complete bridge superstructure in a standard container. The kit is transported to the building site and arrives ready to be assembled in the field to promote rapid construction. The arches can be customized to span up to 65 feet with a rise as low as 20% of the span. Multi radius and multi-span designs are also available.
- The bridge industry can utilize the tools and processes used by other industries to quickly produce and transport products to customers. This paper will show how to apply these techniques to the bridge industry in order to provide quality bridge systems while significantly reducing manufacturing, delivery and construction time. The results are higher quality bridge systems that are cost effective, last longer and are accelerated from concept to completion.
- The Amazon business model can be used as an example of how small to medium span bridges can be delivered faster and more efficiently, consider the following. Amazon creates value for their customers by the synergistic combination of convenience, speed, customization, competitive pricing and reliability of delivery. They are very good at doing this through innovative logistics and attention to supply chain management techniques. By definition logistics is the management of the flow of things between the point of origin and the point of consumption in order to meet customer requirements. Whereas supply chain management consists of all stages involved in fulfilling a customer's request. The supply chain includes raw material suppliers, manufacturers, transporters, installers and even customers themselves.
- The supply chain is most efficient when it is optimized for responsiveness to the market it serves.

- In applying these concepts to the accelerated bridge and prefabricated bridge element market there are six areas where companies position themselves to define their capabilities to provide exceptional customer service and a superior performing product. These areas are: Efficient quality manufacturing, inventory control, location, transportation, customization and installation support.

Bridge in a Box Model

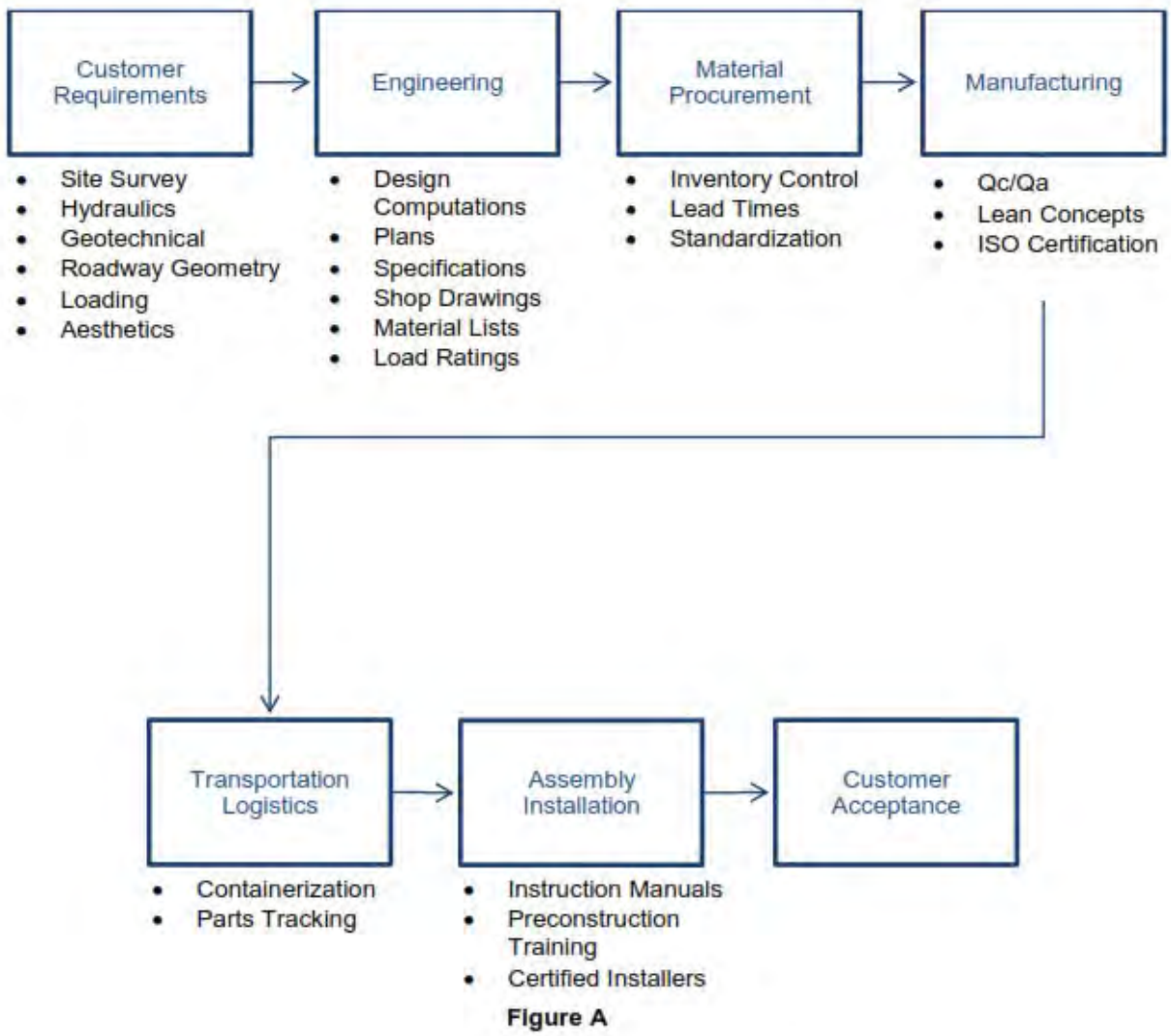
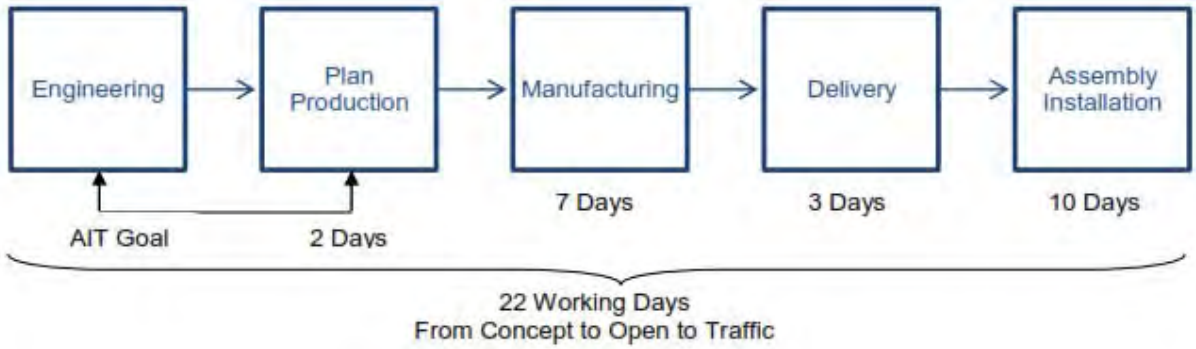


Figure A

Bridge in a Box Product Line Process Goal



Concepts

- Rapidly Deployable Bridge System
- Standard Components
- High Speed Supply Chain Manufacturing
- Custom Engineering Design/ Drawing Production Faster Than Traditional

Figure B

Summary

Accelerated Bridge Construction (ABC) and Prefabricated Bridge Elements are terms that are driving the bridge building business today and into the future. This paper has discussed a prefabricated bridge delivery process and product that offers improvements in delivery and installation that will lead to a faster more efficient process of bridge delivery that is modeled after more commodity based business.

This will require a paradigm shift for owners, engineers, and contractors to accept more standardized, manufactured components then they have in the past, this will lead to a faster more efficient solution to our bridge infrastructure crisis.

EXPERIMENTAL STUDY OF A PRECAST BARRIER WALL SYSTEM PERFORMANCE FOR BRIDGE DECKS

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ABSTRACT

This paper presents a kind of barrier-to-deck connection detail of the precast concrete barrier wall which use the wet joint to expedite construction and minimize traffic disruption.The experimental program was undertaken to investigate the structural behavior and ultimate load carrying capacity of the developed barrier wall and its connection with the bridge deck slab. The group of full-scale barrier wall models were fabricated, which included four specimens to be tested. All of the actual-scale barrier wall models were tested up-to-collapse under different load conditions to examine their static load carrying capacity and crack patterns. The results show that the proposed precast concrete bridge barrier system is as good as the cast-in-place barrier system with respect to ultimate load-carrying capacity.

INTRODUCTION

Bridge barriers are developed to delineate the superstructures edge to reduce the consequences of vehicles leaving the roadway or humans leaving the sidewalk. Concrete barriers appear to be simple and uncomplicated, but in reality, they are sophisticated safety devices.

The use of prefabricated elements and systems in bridge construction has gained interest among transportation authorities. Through mass production of the materials, repeated use of forms, and reduction of on-site construction time and labor, significant economic benefits can be achieved. Work zone safety rapid construction and traffic disruptions are both the fairly important issues to be considered in either new bridge construction or aging bridge replacement. As a result, precast concrete is a potential solution. Precast concrete elements and systems can be quickly assembled, reduce the effects on the environment in the vicinity of the site, and minimize delays and inconvenience to the traveling public, saving time and taxpayers' money.

As such, the objective of this research study is to develop cost-effective, crashworthy, and durable precast concrete bridge barrier system to accelerate the construction of new bridges and the replacement of deteriorated bridge barrier walls. The proposed precast barrier wall system is developed from the castin-place barrier wall used widely in viaducts of ShangHai China, both of this two kinds of barrier system meet the criteria of Specifications for Guidelines for Design of Highway Safety Facilities(Chinese code).

DESCRIPTION OF THE PROPOSED PRECAST BARRIER WALL SYSTEM

The proposed barrier wall system is expected to have the following characteristics (1):

- Precast concrete barriers are certain to have better appearance and quality than cast-in place (C.I.P) concrete barriers.
- Bridge barrier walls can be installed as quickly as possible.
- Protruding reinforcing cages(named “U”) will interfere with the finished bridge deck surface.
- Labor-intensive installation of reinforcement in the field is eliminated.
- No expensive barrier forms are needed.
- The precast concrete barrier wall can be connected with the existing deck slab by drilling into the deck slab and anchor the “U”reinforcing cage to the deck using bar adhesive.
- It may be installed at any time as the C.I.P do.

This investigation resulted in a barrier wall system (Fig. 1) in which barrier walls of 4 m segments were fabricated in the precast concrete plant and shipped to the bridge site(Fig. 2). The vertical connections between segments have been designed while will not be demonstrated in this thesis. The proposed barrier wall was intended to meet the criteria for Guidelines for Design of Highway Safety Facilities (2).

Performance level 3 (PL-3), representing the majority of bridge barriers in ShangHai viaducts.

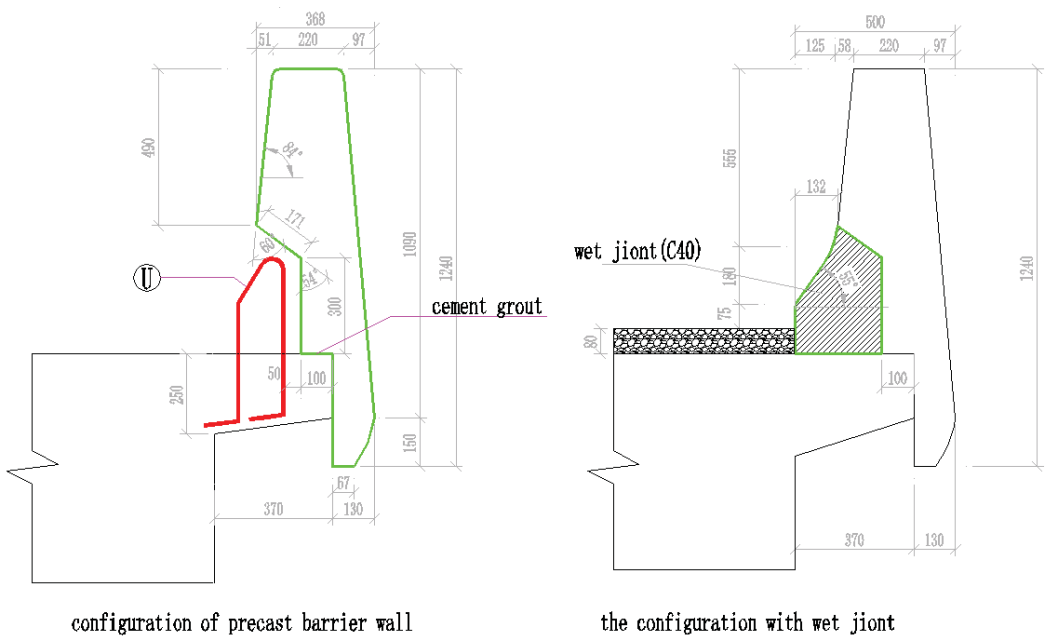

Fig. 1 Structural details of proposed barrier wall system. Note: Dimensions are in millimeters.

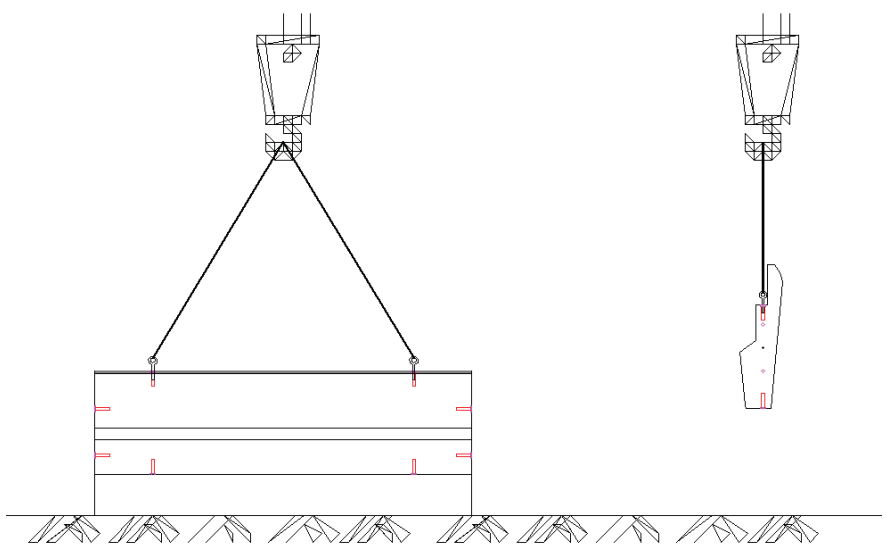

Fig. 2 The way to ship the segments

The depth of the barrier wall from the top of the wall to the top of the concrete deck slab was 890 mm, including 870 mm net depth over the 20 mm thick asphalt layer. The bottom and top widths of the barrier wall were 500 and 220 mm, respectively. A “U” shaped reinforcing cages was embedded in the precast concrete hollow slab. After hardening of the deck slab, an overlay of 20 mm concrete grout was introduced over the concrete deck slab edge with a width equal to the barrier width. Barrier segments were then laid over the deck slab edges, prepared steel support rods were used to avoid the roll of barrier wall before the hardening of the wet joints.

Figure 2 shows the way to ship the precast segments and the overview of vertical connections between segments. While the experiment will only concentrate on the connection between wall and slab.

EXPERIMENTAL PROGRAM

To verify and substantiate the properties of proposed barrier wall system compared with the traditional C.I.P, four full-scale barrier wall models were fabricated and tested to collapse.

Barrier Models

Accordingly, models M1 to M4 (Fig. 3) were erected. Model M1 and M2 were intended to examine the crack pattern and capacity of the two kinds of barrier walls under eccentric and central patch load,while

M3 and M4 were under horizontal uniformly distributed load to investigate the different performance. M2 and M4 were both control specimens representing the monolithic cast-in-place reinforced concrete barrier wall used in ShangHai for performance level 3 (PL-3).

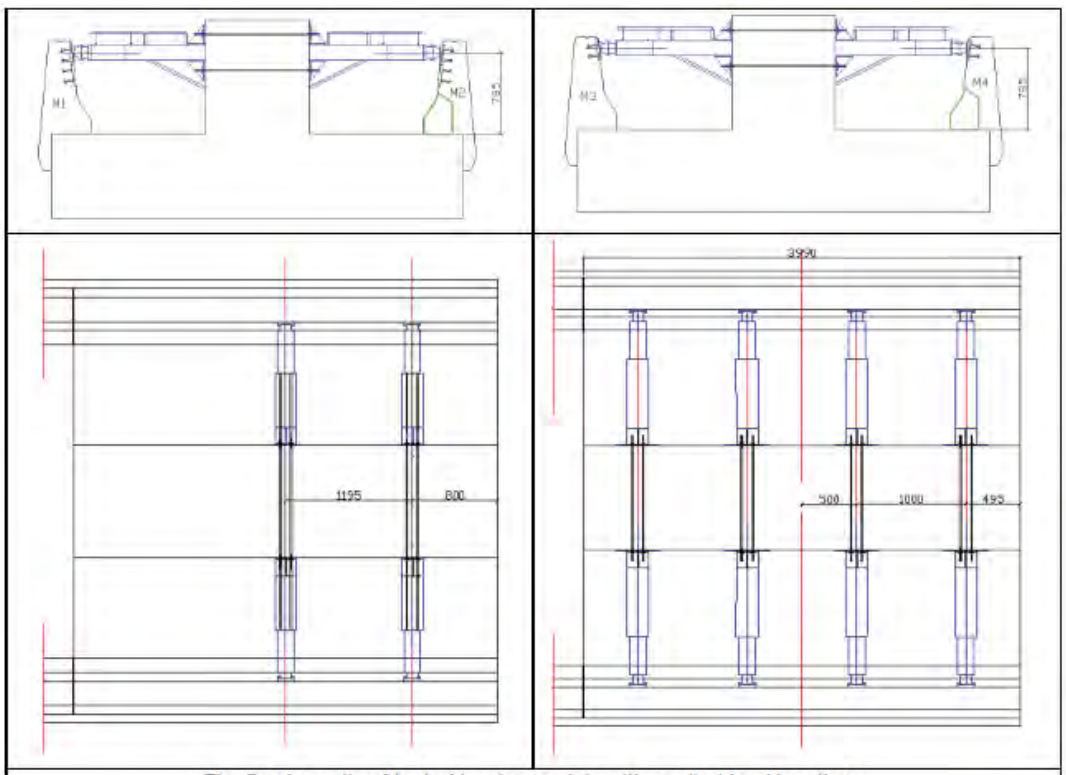

Fig. 3 schematic of tested barrier models with applied load locations

All transverse loads were applied to the inner face of the barrier wall at a 785 mm height measured from the top of the concrete deck slab.

Material Properties

The design of the C.I.P barrier walls dictated using diameter 16mm HRB335 reinforcing steel as the main inner face vertical reinforcement spaced at 150 mm (1 and 2 in Fig.4), while diameter 8mm HRB335 reinforcing steel at the outer face with the same space(5 in Fig.4). The longitudinal reinforcement (3 in Fig.4) and transverse hoops (4 in Fig.4) were R235 diameter 10mm and 8mm, respectively. The concrete using C30.

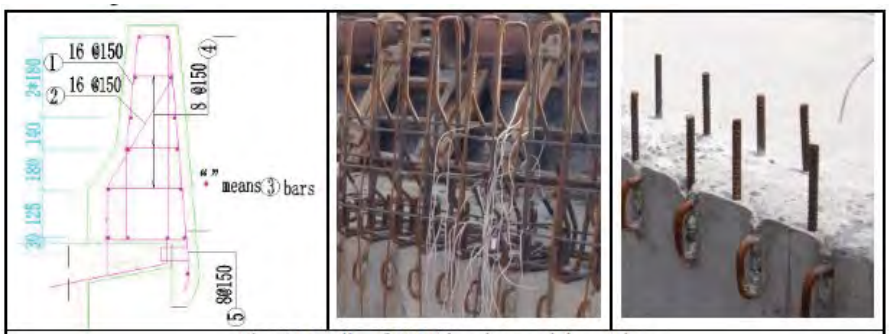

Fig. 4 Details of C.I.P barrier models section

The design of the precast barrier walls dictated using diameter 16mm HRB335 reinforcing steel as all of the main vertical reinforcement spaced at 150 mm(1, 2 and 5 in Fig.5) as the C.I.P do, and some of the longitudinal reinforcement that located in wet joint using diameter 16mm HRB335 reinforcing steel ,too.

(3 in Fig.5). The other longitudinal reinforcement (3 in Fig.5) and transverse hoops (4 in Fig.5) were R235 diameter 10mm and 8mm, respectively. The concrete using C40. It can be investigated that the reinforcement amount of P.C is 40% more than the C.I.P.

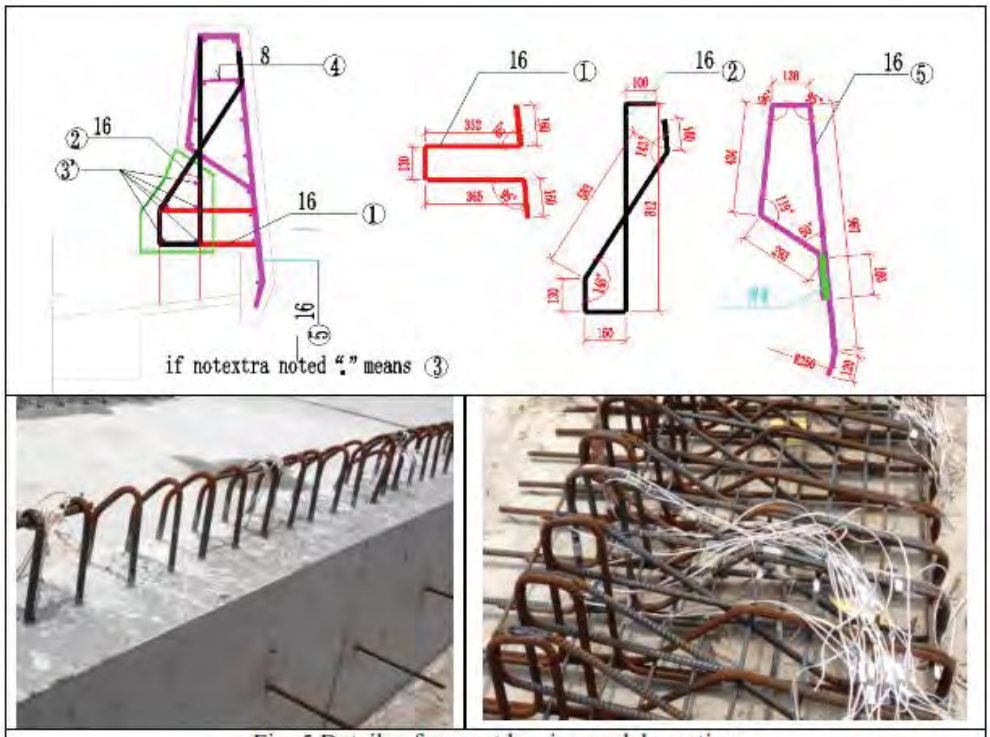

Fig. 5 Details of precast barrier models section

Instrumentation and Test Setup

Strain gauges were mounted on the vertical and horizontal bars along the 4m barrier wall segment. Strain gauges were also attached to the concrete surface at the lowest part of the outer face of the precastconcrete barrier wall. Linear variable displacement transducers (LVDTs) were installed horizontally along the rear of the barrier wall height. (Fig.6)

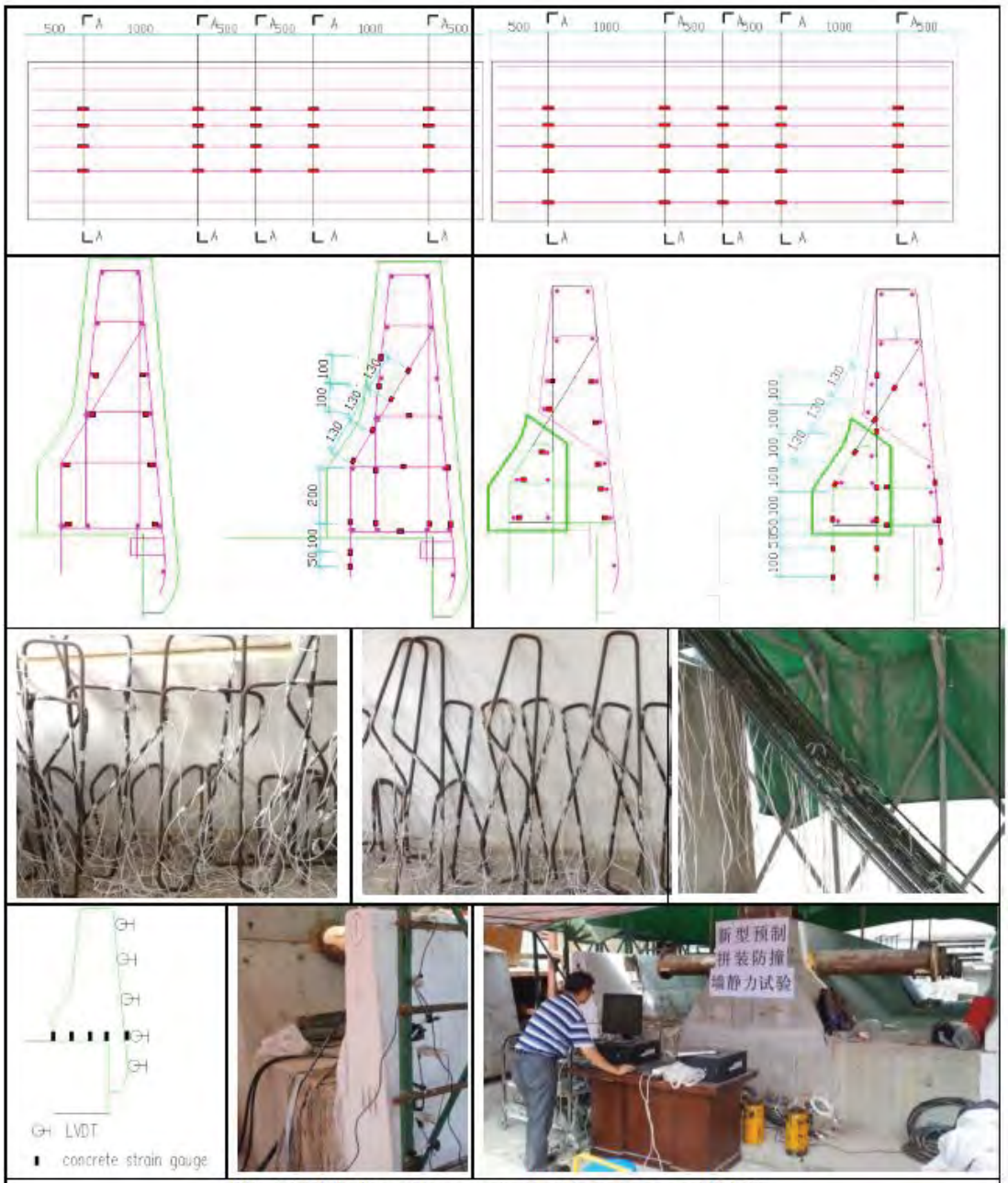

Fig. 6 Distribution of bars and concrete gauges and LVDTs

Test Procedure

According to the criteria of Specifications for Guidelines for Design of Highway Safety Facilities (Chinese code), the PL-3 barrier taller than 0.6m should have the designed lateral load capacity of 430KN every 4m along the wall.

During the test of M3 and M4, 4 jacks applied incremental load to push the wall to collapse. Things were a little different with that of M1 and M2. Jack act on the center line (Fig. 3)apply incremental load to ¼ of designed load then free the pressure, and the eccentric jack followed to push the wall to collapse.

The general load rate is:

1) Pre-press to 10KN

2) Load to 1/4 of designed capacity lateral force;

3) Load to 1/2 of designed capacity lateral force;

4) Load to 3/4 of designed capacity lateral force;

5) Load to designed capacity lateral force;

6) Load to 1.5times of designed capacity lateral force;

7) Load to 2.0times of designed capacity lateral force;

8) Load to 2.5times of designed capacity lateral force;

9) Then every time 1/2 of designed capacity lateral force added to push the wall to collapse.

The load was maintained for about 3 minutes at each increment to observe crack initiation and propagation as well as changes in barrier geometry as depicted from LVDT readings. Failure was reached when the displacement readings from sensors increased without further load increase.

EXPERIMENTAL RESULTS

The experimental program was intended to study the structural behavior and ultimate load-carrying capacity of the precast concrete barrier wall system under static loading. This includes load-deflection and load-strain histories, crack patterns, and failure modes. The results are briefly summarized in the following section.

Crack Pattern and Failure Modes

Barrier model M1:During the load case 1(central patch action), the first crack was observed at the corner of load steel plate appeared horizontally at the load of 215KN. With increasing load, more crack were developed and mostly appeared 45 degree angle with the horizontal line. As the load increase to the designed level 430KN, relief the jack and most of the formed crack closed while some kept open.Then turn to the load case 2(eccentric patch action), the crack developed similar to load case 1, except for the crack will penetrate into the side face and top face of the barrier wall with an obvious open width when the designed load coming (Fig. 7), and more crack were observed at outside of the wall. With increasing

load, the main 45 degree crack through the load plate kept developing and finally lead to the failure, cover concrete on the outside wall spall down and some crack will penetrate into the slab (Fig. 8).

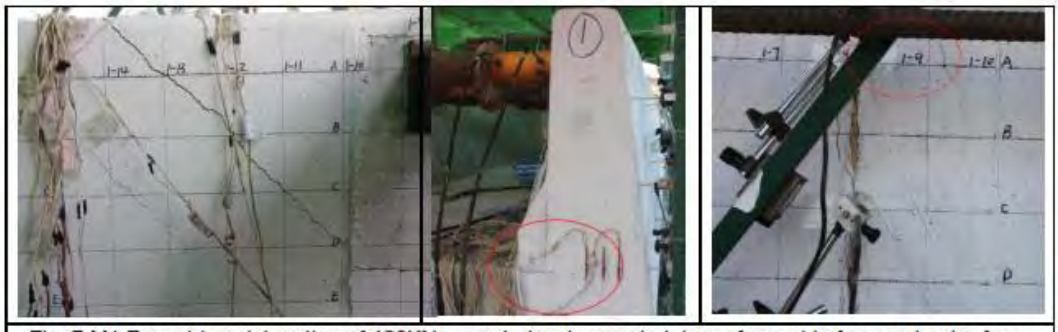

Fig. 7 M1 Eccentric patch action of 430KN : crack development at inner face, side face and outer face.

Fig. 8 M1 Eccentric patch action of 510KN : shear failure of barrier wall and light damage of slab

Barrier model M2:The precast barrier wall performed much better during the load case 1(central patch action), the first crack was also observed at the corner of load steel plate appeared horizontally at the load of 215KN. With increasing load, few crack were developed. As the load increase to the designed level 430KN, relief the jack and all of the formed crack closed. Then turn to the load case 2(eccentric patch action), the crack developed similar to that of M1 with fewer crack amount. Crack between wet joint and precast segment can be observed clearly when the designed load coming (Fig. 9) With increasing load, the main 45 degree crack through the load plate kept developing quickly and lead to the failure,severe crack will penetrate into the slab (Fig. 10).

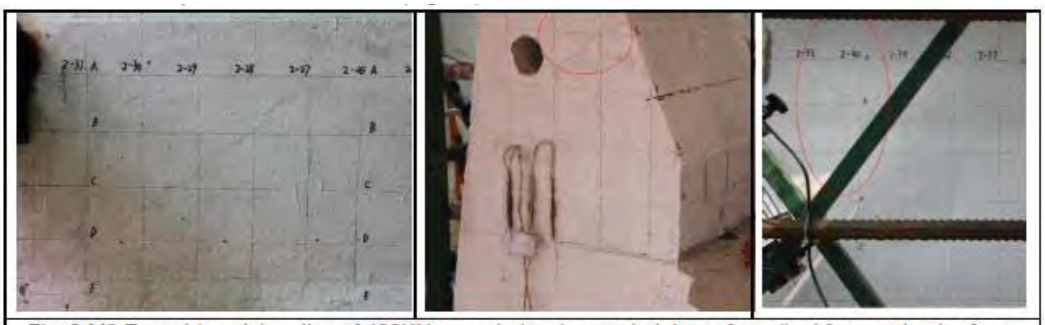

Fig. 9 M2 Eccentric patch action of 430KN : crack development at inner face, jiont face and outer face.

Fig. 10 M2 Eccentric patch action of 540KN : shear failure of barrier wall and damage of slab

Barrier model M3:Four jacks acted simultaneously,the first crack was observed at the section changed height performed a horizontal way across all length at the load of 163KN for each jack. With increasing load, more horizontal crack were developed both inner and out face. As each of jack load increase to

216KN, severe crack were investigated between barrier wall and slab, the crack penetrated though the wall width at the interface level, wall body and slab surface seemed to be tilting. The specimen failure with a maximum displacement of 60mm at the top. (Fig. 12)

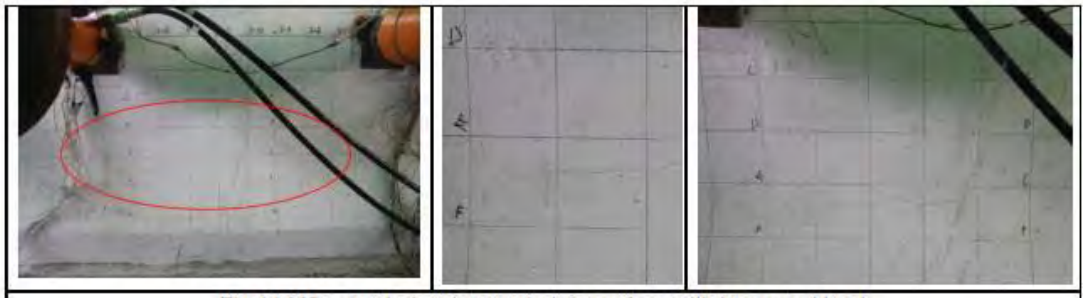

Fig.11 M3 : crack development at inner face with increased load

Fig. 12 M3 : Flexure failure of barrier wall and damage of slab

Barrier model M4: The first crack was observed at the section changed height (top of wet joint) performed a horizontal way across all length at the load of 81KN for each jack, which is earlier than M3.

With increasing load, second crack were developed at bottom of the barrier wall (bottom of the wet joint).

Before failure, the crack still developed around the wet joint. (Fig. 13) As each of jack load increase to

229KN, a severe horizontal crack was investigated in the slab, the crack penetrated though half the slab width from the side view, some concrete crash out at back of the wall, wall body and slab surface seemed to be tilting, too. (Fig. 14)

Fig.11 M3 : crack development at wet joint interface with increased load

Fig. 12 M3 : failure of slab

Load-Displacement curve

Load-Displacement curves shows in Fig.13 and Fig.14, the lateral force represent the load of one jack.

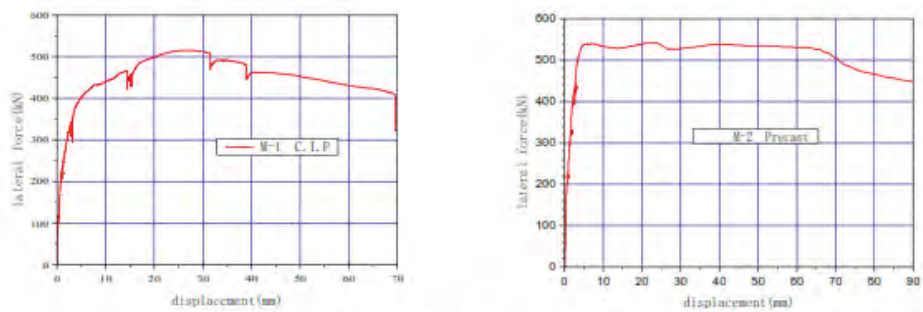

Fig.13 Eccentric patch load case L-D curve

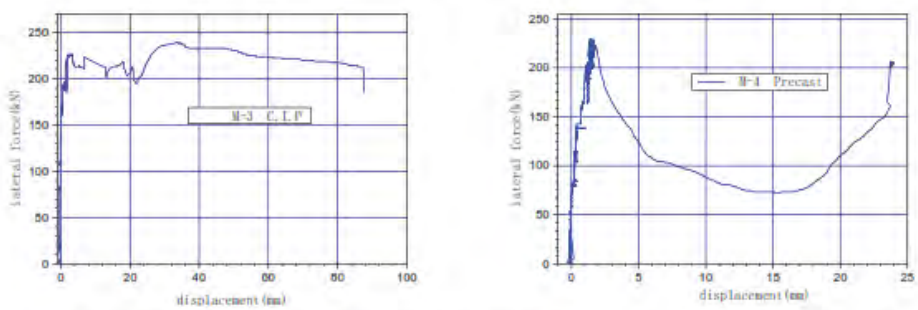

Fig.14 Uniform load case L-D curve (some trouble with the LVDTs of M4 during test)

CONCLUSION

Based on the data generated from experimental tests, the following conclusions can be drawn:

- M1 of C.I.P barrier wall and M2 of precast barrier wall both failed due to shear failure of wall body under eccentric patch load case associated with secondary combined flexure-shear-bond crack patterns appearing in the deck slab;

- M3 of C.I.P barrier wall a failed due to flexure failure of wall body at the interface of wall and slab under uniform horizontal load case associated with severe flexure-shear-bond crack patterns appearing in the deck slab;

- M4 of precast barrier wall a failed due to severe flexure-shear-bond crack patterns appearing in the deck slab under uniform horizontal load case and the wall body keep monolithic;

- The proposed precast concrete barrier wall system proved to be as good as the cast-in-place concrete barrier system and adequate strength based on Guidelines for Design of Highway Safety Facilities(Chinese code).

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ACKNOWLEDGMENTS

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G-13: ABC CASE STUDIES

SALMON RIVER BRIDGES

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ABSTRACT

Two Bridges in central Idaho, one over the Salmon River and one over the East Fork of the Salmon River, were replaced in the summer of 2014 using precast bridge elements in order to accelerate construction and complete this replacement project in one construction season (April thru September). Constructed conventionally this project would have historically taken two years to complete. Because the long winters in this region prevent year round construction the Idaho Department of Transportation was interested in completing this project in 6 months or less. The precast elements that were used in this project including abutments, pier columns, girders, and deck made the shortened schedule possible.

INTRODUCTION

The old bridges over the Salmon and East Fork of the Salmon were originally designed and constructed by the Bureau of Public Roads, U.S. Dept. of Agriculture in 1937. The bridges cross these two rivers just upstream of their confluence. Both bridges carry SH-75 and are separated by only 350'. The old bridge over the main Salmon was a 240-foot four span steel plate girder bridge, while the bridge over the

East Fork was a 168-foot three span steel plate girder bridge. Both bridges had a curb-to-curb width of 24 feet.

PROJECT DEVELOPMENT

For many years the Idaho Transportation Dept. has been planning the replacement of these two bridges.

The bridges' age, structural condition and narrow width required improvements, however the rugged area and sensitive nature of the river and its habitat for salmon required a lot of study of various alternatives.

Realignments of the highway and other crossing sites were studied but it was ultimately determined that replacing the old structures on an alignment adjacent to the old just to the south would have the least impact to both the environment and to traffic on SH-75. The new alignment was offset from the existing highway by 42 feet, just enough to build the new bridges without interfering with traffic on the existing

bridge. Due to the long winters in this area at an elevation of 5400' above sea-level, and the sensitive nature of the salmon spawning habitat in the river the project needed to be built in a 6 month window between April and October. The span over the East Fork could be spanned with a 137 foot single-span

prestressed girder bridge; however the 240 feet across the main Salmon required multiple spans with a precast concrete girder bridge. This could have been accomplished with a two-span bridge; however that would have put the single pier near mid-channel and would have perpetuated the problem of putting an obstacle in the river that would collect and backup the large amount of ice and other debris that is carried by the river. A 3-span configuration was selected to keep the piers near the banks for easy construction access, minimize disturbance to the spawning habitat and provide an unobstructed channel for ice and debris. The biggest impact to the river itself was actually caused by the removal of the three existing piers in the channel after the completion of the new bridges.

Deck Design

Historically one of the most time consuming tasks in bridge construction is forming, tying rebar, and placing, finishing and curing of the concrete bridge deck. This is one area that had a significant potential for time savings by using precast deck panels. At the time of project development Idaho had only used precast deck panels a couple of times and never on a new bridge were the deck would need to be composite with the girders. In order to limit the use of cast-in-place concrete in closure pours we wanted to use full width deck panels but we were concerned about how to handle the crown in the deck cross section.

We wanted to keep the geometry of the panels as simple as possible to facilitate the use of prestressing in the transverse direction as well as post-tension them longitudinally. We eventually settled on flat deck panels and decided to make up the crown section with an asphalt overlay. This decision resulted in an overlay that varied in thickness from 0.2' at the curb line to 0.6' at centerline, a considerable increase in dead load on the structures which we deemed to be worth it. The East Fork River Bridge had a skew of 15 degrees and we decided that this was a small enough skew the deck panels could be constructed on the skew and therefore simplify the design by using a single panels detail for all the panels on the bridge. However the Main Fork Bridge had a skew of 26 degrees, too much to cast the deck panels on the skew so the panels were squared up and the end panels were cast in a triangular shape to make up for the skew. This complicated the details somewhat but overall resulted in a better design. Both bridges were designed without expansion joints and the girders on the Main Salmon River Bridge would be constructed continuous and therefore the deck also needed to be continuous over the piers. This was accomplished by simply laying out the panels along the length of the bridge without regard to the locations of the panel

joints in relation to the piers. In order to ensure composite action between the deck and the girders pockets where designed into the panels to accommodate weld studs that could be installed in the field by welding onto steel plates that would be embedded into the top flange of the girders. The deck units were conventionally reinforced based on traditional AASHTO bridge deck design procedures. The deck was designed to be 8.5” thick, with a top cover of 2.5” and bottom cover of 1” and with epoxy coated reinforcement in both top and bottom mats. The reinforcement pattern varied due to the locations of the weld stud pockets but on average the top and bottom mats consisted of #6 bars at 6” each way. Because the shear pockets are 12” by 11” and are spaced every 24” only 12” of deck are available to fit four #6 bars both top and bottom. If we had put all the bars in the spaces between pockets the actual spacing between bars would have been 3”. Thinking that was a little tight we decided to place one bar in the center of the duct pocket. That increased the spacing to 4.5” in the area between pockets and resulted in a spacing of 7.5” within the pocket, a much better distribution of reinforcement. In the longitudinal direction the panels were designed to be post-tensioned with a net compression force of about 350psi across the full width of the panels. This was accomplished using four ½”-strand tendons in 2” ducts spaced at approximately 36”. The tendons were placed at mid-depth of the slab. The main purpose of the post-tensioning was to prevent transverse deck cracking, especially in the areas over the piers were the deck is continuous. The riding surface would be asphalt which was placed on a liquid applied waterproof membrane.

Girder Design and Details

The concrete girders were conventional bulb-tee girders with a depth of 72”. They were designed in accordance with AASHTO LRDF Specifications assuming the deck panels act compositely with the girders. The main difference between girders that are designed and detailed for a cast-in-place deck and a precast deck is the method of providing shear reinforcement across the girder/deck interface. Typically the vertical shear stirrups in the girders are extended into the deck and the deck is cast around the stirrups. In this case steel plates were imbedded in the top flange every 2 feet and weld studs installed through the pockets in the deck panels.

Pier Cap Design

Initially the pier cap was planned to be precast with the other bridge elements however by using a pier design with a single column the large cap that was needed became too heavy to ship to the site.

Consequently the pier cap was designed as a conventional cast-in-place hammer head pier cap.

Pier Design

The Salmon River carries a fair amount of ice and debris and it is the general policy of ITD to not construct multiple column piers in rivers to preclude the debris build up that often occurs between columns. Also in order to aid construction with as light of a column as possible a relatively small 3’ by 9’ 6” rounded end column shape was selected. The foundations are conventional cast-in-place pile caps on a group of 28 H-piles. The controlling design load was the lateral force generated by 6’ thick ice. The method selected to attach the precast columns to the foundations was grouted sleeve connections. The grouted sleeves were designed to be located in the columns and about 10 inches of rebar was left extending from the top of the foundations. In hindsight we would have placed the grouted sleeves in the foundation pile cap and the grouting operation would have been somewhat easier and the column details would have been simplified by not having to detail two different diameter spiral reinforcement cages, it would also have eliminated the excess cover over the main vertical bars that was needed to accommodate the large diameter grout sleeves at the base of the column.

Abutment Details

The abutments for both bridges were designed as integral with the superstructure and supported on a single row of eight H-piles. The lower half of the abutments were precast with 30” round voids every 5’ to receive the piles and the upper half above the beam seats was cast-in-place to incorporate the lower abutment, girders and precast deck panels into one monolithic structural unit. When the structure was complete a cast-in-place approach slab was installed at the ends of the bridges.

CONSTRUCTION

During construction the first problem to arise related to the precast elements was the difficulty of maintaining the correct location of the driven H-pile. The soil is very rocky with some fair sized boulders present below the surface. The piles would start out in the correct position but once they were driven to about 20 or 30 feet they would often strike a boulder and move laterally up to about 12 inches. With a 14 inch pile (20 inches on the diagonal) there was only a 5 inch tolerance for being out of position. After the first couple of piles were driven and then pulled back out of the ground due to the misalignment we sat down with the Contractor to discuss the problem. We decided that rather than pre-drilling a hole at every pile location, which would have been expense and time consuming, we came up with the idea of providing a driving template to keep the piles aligned. The template was created with sections of H-pile welded into a ladder configuration that was then buried in the ground at a level just below the bottom of the precast abutment sections. Once the template was in place all eight piles at each abutment where driven an initial 10 feet or so and then each pile was driven to its full depth. The template worked just as

planned and every pile ended up within a few inches of the correct position. The template was left in place and the precast sections where set with no other problems. We had experienced this same problem to a lesser extent on previous projects that utilized precast pile caps and we now plan to specify this method in future projects whenever this problem is anticipated.

SUMMARY

The overall project was a success in that all our objectives were met:

- Two lanes of traffic were maintained on SH-75 throughout the duration of the project.
- The work was completed in less than 6 months.
- The new bridges are functioning well and we expect at least 75 years of service. Similar to the bridges that were replaced.
- The work did not exceed the project budget of $7.5 million. However there were a few things learned that will be corrected in future projects of this type.
- The grouted splice sleeves will probably be designed to be in the footing rather that the column.
- Use of templates to ensure the alignment of the driven piles will be specified on future projects.

12 DAY BRIDGE REPLACEMENT PROJECT IN VERMONT INCLUDED A 47 MILE DETOUR

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INTRODUCTION

Due to a lengthy detour route this Accelerated Bridge Construction (ABC) project utilized prefabricated bridge elements to replace a structurally deficient bridge in only 12 days. The Vermont Agency of Transportation (VTrans) elected to replace the existing bridge due to its poor condition. The 101' long replacement bridge utilized prefabricated bridge units (PBU's), which consisted of modular weathering steel plate girders connected by a preconstructed concrete deck. The Contractor was allowed to self-perform the deck construction on the PBU's in a nearby staging area, which reduced the PBU costs by more than 20% when compared to off-site fabrication and transportation.

EXISTING CONDITIONS

The structure's remote site location, and the required 47 mile detour for thru traffic on this heavily travelled corridor in Vermont was a major factor which dictated the use and type of accelerated bridge construction (ABC). . The existing structure was an 85' simple span structure with a 20 degree skew.

National Forest Land on either side of the bridge sits in close proximity to VTrans Right-of-Way. In order to minimize permit and right-of-way requirements for this project, the design team had to develop details to minimize the environmental footprint of the project.

PRELIMINARY DESIGN

Preliminary design included an initial public outreach by the project team during the early stages of preliminary design. Through discussion with local residents and businesses, VTrans committed to minimizing environmental impacts along with minimizing the length of time motorists would be required to use the 47 mile detour. In addition to being a major route through the National Forest, this bridge sits on a popular biking route as well. /Several ABC and conventional construction alternatives were developed and evaluated with the primary goal to maintain continuous access to the National Forest or

minimize road closure using accelerated bridge construction with a signed detour. A summary of the Alternatives evaluated is provided below:

Option	Preliminary Cost	Preliminary Total Construction Duration
Full Road Closure with ABC	$1,512,000	3 Months (2 Week Bridge Closure)
Phased Construction with Temp. 1 Lane Bridge	$1,750,000	18 Months
Full Bridge Closure with 1 Lane Temp. Bridge and Temp. Signal	$1,971,000	18 Months
2 Lane Temporary Bridge	$2,150,000	18 Months

Due to the site constraints, the lengthy detour, and VTrans's familiarity with the use of Prefabricated Bridge Units (PBU's), steel beam PBU's were determined to be the optimal superstructure replacement option for ABC construction. Decked-concrete beams were considered by the design team but the proposed 101' span length precluded concrete due to the heavy beam pick weights and limited right –of-way available for crane placement.

Based upon the results from the preliminary alternative analysis, and through the input provided during the initial public outreach, the full road closure with ABC option was chosen by the design team to be advanced to final design. This option provided both the lowest cost and shortest time to construct. A preliminary construction duration schedule was developed which determined that a 14 day road closure duration would be possible for this ABC project.

FINAL DESIGN

The PBU's were designed following AASHTO criteria. The longitudinal closure pour connection between the PBU's was designed and detailed by VTrans staff for other PBU projects throughout the state, with the connection made with a fast early-set concrete.

The subsurface investigation program (boring program) was performed in the early stages of final design.

The results of the program showed that bedrock was at a sufficient depth to allow for an integral abutment, which is VTrans's preferred substructure type. The design team performed a industry-wide scan of the integral abutment system to determine if there were any details that could be used in conjunction with PBU's to accelerate the road closure portion of the project. The hinged-integral detail was discussed, and based on a review by the design team, the following advantages of this detail led to the approval of the hinged integral abutment for this project:

- The hinge eliminates superstructure rotation which is typically transferred to the piles in integral abutments.

- The anchor bolt and bearing plate detail provide a good detail for placement of the PBU's during construction, which allows for greater construction tolerance during construction.

- The number of girders required (8), resulted in 16 anchor bolts being required, which provided sufficient shear resistance across the precast abutment joint.

The precast abutments were designed following the latest details provided by the SHRP 2 ABC Toolkit for a pile cap supporting PBU's. The pile caps were designed and detailed with transverse post tensioning that ensured adequate contact stress only. VTrans does not require grouting of the transverse post tensioning ducts when they are not used as primary reinforcing, therefore post tensioning was eliminated to save time.

During final design, a special provision was developed which allowed for the PBU's to be fabricated by the Contractor (self-performance) prior to the road closure. Previous VTrans projects had required PBU fabrication by a PCI Certified Facility. Self performance by the Contractor was permitted in order to reduce overall project costs and reduce project coordination and the required distance the PBU's must be transported.

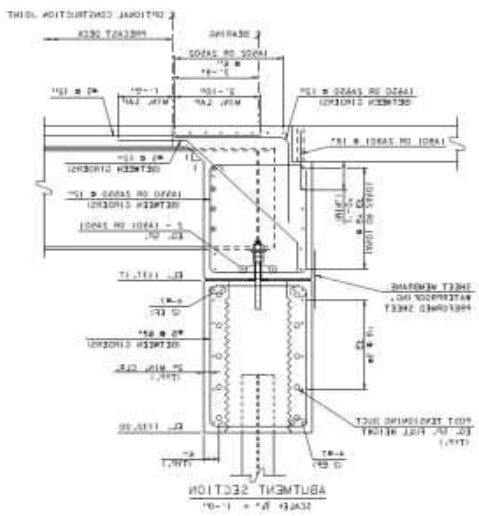

Figure 1: Hinged Integral Detail

Early on the design team identified that driving of the piles posed a major risk to the completing the project within the agreed to 14 day road closure goal. Any obstruction encountered above the minimum design pile elevation during driving time would require additional time to core through or remove. Because of this, the piles were strategically located during design so that the Contractor could pre-auger holes for the piles in advance of the road closure while maintaining one lane of alternating traffic.

CONSTRUCTION

The project was advertised in October of 2013, with bids received from eight General Contractors. The winning bid was submitted by The Luck Brothers Inc. at $1.64 million (5% above Engineers Estimate of$1.56 million).

Picture 1: PBU Fabrication

Prior to the road closure, the pre-augering was performed by the Contractor using alternating one-way traffic with flaggers.

Several unexpected obstructions were encountered during the pre-augering including fill concrete leftover from construction of the existing abutment.

Substantial drilling and coring was required to penetrate through the obstructions.

The Contractor elected to selfperform construction of the PBU's which proved to save cost. Luck Brothers' bid price for the

PBU's (which included all steel and concrete superstructure components) was 80% of similar PBU unit costs on other VTrans projects. A staging area was located several miles from the site, and the girders were erected on temporary supports which mirrored the final site conditions. The longitudinal deck joints between PBU's were formed and blocked out, and the deck was placed and cured in a similar manner as to how a conventional bridge deck would be placed.

Closure of the road occurred on August 7, 2014. The PBU's were erected using a skid beam which spanned both abutments, and a dual-pick sequence determined by the Contractor. The dual-pick sequence minimized required tree clearing and did not require any additional right-of-way by the Contractor. The overall road closure duration was 12 days, 2 days ahead of the 14 day contract allowance. The road was reopened to traffic on August 18, 2014 with full overall construction completed in October, 2013.

LESSONS LEARNED

Several valuable lessons were learned on this challenging project that can be applied to future ABC projects, which include:

- The hinged integral abutment detail can be used in conjunction with ABC projects, and is especially suitable to precast integral abutment bridges which have PBU superstructures.

- Locate piles during design so that pre-augering to clear potential obstructions prior to the road closure greatly reduces the time-risk element on ABC projects with deep pile-supported foundations.

- Contractor self-performance of the PBU's is possible and can significantly reduce overall project costs.

CONCLUSION

The hinged-integral abutment detail is a practical detail that works well in conjunction with ABC projects, particularly projects with PBU's and precast concrete abutments. In addition, careful planning in pile design can mitigate project risks. This project clearly showed that overall project costs and construction duration can be significantly reduced through careful planning,

NEW JERSEY DEPARTMENT OF TRANSPORTATION (NJDOT) ROUTE 18 BRIDGE OVER ROUTE 1 – SUPERSTRUCTURE WIDENING AND REPLACEMENT

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INTRODUCTION

Under contract to the New Jersey Department of Transportation (NJDOT), HNTB provided final design and construction engineering services for the roadway/bridge widening and superstructure replacement at the NJ Route 18 and US Route 1 interchange in New Brunswick, Middlesex County, New Jersey. This interchange is one of the most congested in the State as it involves traffic from two high volume State highways and is a major connector to the NJ Turnpike.

The primary goals of this project were to replace the existing deteriorated bridge deck slabs and to extend the existing three-lane section of Route 18 Northbound north of the interchange into the project limits, thereby improving operational efficiency. This project minimized impacts to these high-volume roadways by limiting the most disruptive construction activities to summer weekends when traffic volumes and activity on the nearby Rutgers University campus are at their lowest. Accelerated Bridge Construction

(ABC) techniques were utilized to make the construction activities possible in limited construction windows.

PROJECT DESCRIPTION

The interchange, originally constructed in 1959, includes a two-span bridge carrying Route 18

Northbound (NB) and Southbound (SB) over Route 1 and a single-span bridge carrying Route 18 NB over the ramp from Route 1 SB to Route 18 SB (Ramp E). Based on the age and condition of the bridge, the owner designated the bridge deck slabs to be scheduled for replacement.

With traffic volumes on Route 1 and Route 18 each exceeding 115,000 vehicles per day (VPD), NJDOT could not allow permanent lane closures for typical staged construction because of the long queues this would cause on a daily basis. The site is located only ½ mile from NJ Turnpike Interchange 9 and is

immediately east of the Rutgers University campus, and is an important local link with no convenient detour.

Due to these heavy traffic conditions, HNTB developed a staging scheme that utilized ABC elements to enable superstructure replacement within 57-hour weekend construction windows (Friday night through Monday morning), thus minimizing impacts to motorists. These ABC techniques included the use of moveable construction barrier, Prefabricated Superstructure Units (PSU), and various construction details as described below.

The project also included the widening of Route 18 NB from the vicinity of Weston Mill Pond to the exit ramp for Route 1 SB. The existing hammerhead pier columns were retrofitted by reconfiguring them as wall-type piers. The newly widened portions of the pier and abutments were founded on micropiles.

The roadway approach and bridge substructure widening work was accomplished during normal weekday lane closure hours. The weekend demolition and erection of the PSUs occurred during the summer of 2015, with the work being confined to a maximum of 10 weekends from May to August 2015.

Other work on this project included the installation of precast approach slab panels, installed during weekday nighttime lane closures, and the reconstruction of the retaining wall-supported ramp connecting

Route 18 NB with Route 1 SB to improve the safety of the ramp by completely reconstructing with larger radii.

ABC DESIGN DETAILS

The project objective required rapid construction during limited weekend lane closure windows.

Additionally, a prescribed minimum number of lanes, in both directions of Route 18 and Route 1, had to remain open during weekend construction. The contractor would incur Lane Occupancy charges if the full roadways were not open by 5 AM Monday morning. To best improve the chances to complete construction and open the roadway to traffic by the required time, consideration had to be given during design to minimize risks that could cause delay.

Thus, it was crucial to give the contractor flexibility and allow for field tolerance to ensure fit-up; misalignment of the structure was a major potential risk of critical delays. Details had to be developed that would provide durability and quality but could be installed rapidly. The following ABC design details were among those utilized for this project.

The existing structure was carefully surveyed during design and again during construction. These surveyed dimensions were reviewed and then utilized when detailing the prefabricated elements such as

PSUs, precast approach slabs, and bearings.

Moveable Construction Barrier

During each of the 10 weekends, the owner required that two lanes had to be maintained in both directions on Route 18. However, two large cranes were needed for superstructure demolition and installation of the PSUs and the contractor had to rapidly establish sizeable work zones on each side of the bridge. This was accomplished by utilizing moveable construction barrier, which was pre-deployed in the shoulder and moved into place with a transfer machine, in conjunction with median crossovers. The establishment of a contraflow lane allowed for two lanes to remain open in each direction while weekend construction was taking place. The moveable construction barrier was deployed in less than thirty minutes, compared with the estimated two hours if typical temporary construction barrier had been placed, thereby minimizing the overall time needed for setup and breakdown of traffic control.

Prefabricated Superstructure Units

Each PSU consisted of two simply-supported rolled steel beams composite with a precast deck slab, fabricated in the shop and delivered to the site for immediate installation. Units ranged from 55’ to 90’ in length and from 8’ to 13’ in width.

The specifications required that adjacent units be match-cast in the fabricator’s shop. Each entire bridge span was constructed at once to best replicate the surveyed geometry in the field, including horizontal and vertical geometry. The relative bearing seat elevations were set to simulate the actual pier and abutment bridge seats. The deck panels were cast side-by-side, with the longitudinal deck joints blocked out. The match-casting allowed for the bridge to be built at once, ensuring that the PSUs would align with each other during actual installation. Temporary midspan supports were included by the fabricator in the shop to minimize differential camber between adjacent units while the deck slabs were poured.

Due to their size, the units were very heavy, and measures had to be taken to minimize their weight where possible to facilitate transportation and erection. To avoid problematic leaking from bolted-down precast parapets, the parapets were cast integrally with the PSUs in the shop, except at the heaviest unit.

To help compensate for that additional weight, the sidewalks on the bridge were cast in the field. To further minimize weight, new backwalls were not precast as part of the PSUs since the existing backwalls were determined to be in acceptable condition.

Elastomeric Bearings on Steel Bolsters and Leveling Nuts

The PSUs were supported on steel-laminated elastomeric bearing pads. Elastomeric bearings were selected as a seismic retrofit strategy to upgrade the existing steel rocker bearings. Since the profile of the new steel beams was shallower than the existing beams, there was an elevation differential that had to be made up by raising the bridge seat. In order to accomplish this in the limited construction window, the elastomeric bearings were supported on steel bolsters. The bolsters, built-up from thick welded steel plates, were used since the vertical height adjustment needed at some locations was too shallow to use precast concrete. Steel was again used at the taller bolsters for uniformity throughout the project.

The elastomeric pad was vulcanized to the steel load plates, which were shop welded directly to the bolster to minimize field welding. Each bearing and bolster was delivered to the site as one unit, ready to install. Each bolster was secured in place by four anchor rods grouted in core-drilled holes in the pier/abutment bridge seat with high-early strength cement grout. Once the grouted core holes had cured for a prescribed length of time in order to achieve the required strength, the bearings were installed. The masonry plate at the base of each bolster was supported on leveling nuts to allow for vertical field adjustment, similar to what is commonly used for sign support structure base plates. Once the PSUs were in place, the nuts could be adjusted as the unit was being jacked to allow the unit to be set to an accurate vertical alignment. Then a high-early strength flowable cement grout pad was installed beneath the masonry plates. The 3” thick masonry plates were designed to span between the leveling nuts. The anchor rods were designed to act as unbraced columns in compression, able to support vertical and lateral loads independent of the grout pad. The various plates of the bolsters were designed to provide fixity or guided expansion movement, as required by the specific bearing location.

Precast Concrete Plugs in Deck Joint Blockouts

The deck joints were expected to be installed during weeknight lane closures, and thus had to be able to be installed rapidly, while accurately lining up across the opening between the PSUs and/or approach slabs. Thus, the contractor needed some installation tolerance to allow for some adjustment vertically, and horizontally. For a typical bridge, concrete blockouts could be utilized, but the deck joints would not be able to be installed during the weekends until all adjacent units on either side of the joint is installed. A proper riding surface must be maintained; thus, the blockouts would have to be filled in until all adjacent

PSUs were in place along the joint. The blockouts could not be filled with temporary asphalt, which leaves a residue on the concrete that would act as a bond breaker with future concrete. Thus, a detail was developed to temporarily fill the blockouts with precast concrete “plugs”.

These precast plugs allowed for rapid field installation of the deck joints. Blockouts in the PSUs were formed in the shop utilizing wax paper bond breaker. The plugs were bolted down to a precast concrete "shelf" below. The bolts were unfastened and the plugs were removed in the field just prior to the deck joint armoring and support plate installation. During construction, the contractor revised this detail slightly so that the deck joints could be installed during the weekends when he installed the approach slabs.

Polyester Polymer Concrete

Polyester Polymer Concrete (PPC) was utilized in the field as the permanent longitudinal deck joint material. It is typically used for overlays of roadway surfaces where rapid setup and strength is required to allow the road to open to traffic quickly. PPC sets up very rapidly and achieves over 6,000 psi in only 3 hours. This was verified by compression strength testing on 2" cube samples.

The contractor mixed the PPC on site to install it rapidly before it could set up. The PPC has no water used in its mix at all; absolutely no moisture is allowed during installation. Therefore, some schedule float must be available to allow the contractor to reschedule installation in the event of rain, unless tenting could be utilized. The PPC has the consistency of wet sand and was installed by hand. This made it difficult to verify there are no voids; the contractor had to take measures to provide a quality installation.

The aesthetics are somewhat undesirable; the PPC is a brownish-yellow color and can't be tinted lighter to match the surrounding grey concrete, therefore creating the illusion of stripes on the roadway, which could potentially confuse drivers if the longitudinal joints don't match up with the lane lines on a long viaduct.

Precast Approach Slabs

Precast concrete panels were utilized to construct the approach slabs to allow them to be installed piecemeal during daily weeknight lane closures. The contractor revised some of the approach slabs to be constructed using cast-in-place high-early strength concrete during weekend construction windows. This was accomplished using the same traffic control (including the moveable barrier) as the bridge weekend construction, but could only be done on the four-lane sections of Route 18; thus the approach slabs for the bridge over Ramp E were still constructed using the precast slabs.

WEEKEND CONSTRUCTION

Beginning at 8:00 PM each Friday night, the contractor deployed the moveable barrier and the rest of the maintenance and protection of traffic (MPT) devices to establish the work zone. Work continued with the

demolition of a portion of the existing bridge superstructure. On Saturday, bridge seat preparation work was done and the bearings were installed. On Saturday evening, the PSUs were installed, two or three units each weekend. The PPC longitudinal joints were installed on Sunday morning, along with diaphragms and temporary deck supports installed from Route 1 below. Sunday afternoon saw the breakdown of the MPT. With each succeeding week, the contractor's team became more adept at the work; each of the ten weekends finished on Sunday, ranging from 10:00 AM to 7:00 PM, well before the required finish time of 5:00 AM Monday.

BENT CAP DESIGN FOR IH 635 MANAGED LANES PROJECT

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ABSTRACT

The IH-635 Managed Lanes Project (LBJ Express) must be built within the existing facility and keeping the existing traffic which poses significant challenges from the view point of maintenance of traffic, construction phasing and schedule. All of this has required an extraordinary effort in planning and design, incorporating innovative ideas to the design of the structures in order to improve schedule, adapt the structures to the traffic control plan, reduce traffic detours and improve safety during construction.

Different designs were adopted to try to solve each specific case challenge posed by the project. This paper describes some of these innovative designs and how they were implemented during construction.

INTRODUCTION

The IH-635 Managed Lanes Project (LBJ Freeway) will rebuild one of the busiest and most congested highways in North Texas by 2016. This project is one of the largest private-public partnerships undertaken in the United States in terms of complexity and investment value.

The most significant part of the project consists in a full reconstruction of IH-635 general purpose lanes from IH-35E to US-75 and adding six managed lanes in a depressed section for a total length of nine miles. The depressed managed lanes are excavated between retaining walls of approximately half the width of the general purpose lanes on a bridge overhang above the depressed managed lanes supported by straddle bents across the whole width of the six managed lanes.

The fact that the new managed lanes must be built within the existing facility while keeping the existing traffic, with more than 250,000 vehicles driving through the corridor every day, poses significant challenges from the view point of maintenance of traffic, construction phasing and schedule. All this has required an extraordinary effort of planning and design, incorporating innovative ideas to the design of the structures in order to improve schedule, adapt the structures to the traffic control plan, reduce traffic detours and improve safety during construction.

Figure 1. LBJ Express Plan

BACKGROUND. MAIN OBJECTIVE

Most of the design solutions for the structures were chosen to minimize disruption to traffic and construction time. Due to the fact that a large number of the structural elements have similar characteristics, precasting was something that seemed the best option from the very beginning of the design process. The option to "industrialize" the construction process with precasting yards on-site is always an excellent idea to accelerate construction if the conditions are adequate as they were for this project.

Precast elements such us concrete pre-stressed beams, concrete box girders, precast concrete panels were used whenever possible. Following this scheme, precasting the bent caps of the structures was a concept very present from the very beginning of the design but it originated certain problems depending on the cases encountered throughout the project. That the reason why bent cap design is where some of the most innovative ideas have taken place.

DESCRIPTION OF THE SOLUTIONS IMPLEMENTED

Description of the Project

TxDOT awarded this Comprehensive Development Agreement "CDA" project to a Cintra-led Developer consortium, following a competitive process. This innovative 13.5-mile project addresses increased traffic demands through a multi-level highway system. Construction cost is approximately $2 billion, with $2.2 billion total project investment with a concession period of 52 years.

The project has two distinct sections:

1. Express toll direct connectors along IH 35 E, connecting with IH 635 easterly. This section, on a North to South alignment along IH 35 E consists of an elevated direct connector carrying four new toll lanes (with ultimate expansion to six) on long viaducts to avoid congestion on the existing IH 35E general purpose (non-tolled) lanes (Figure 1).

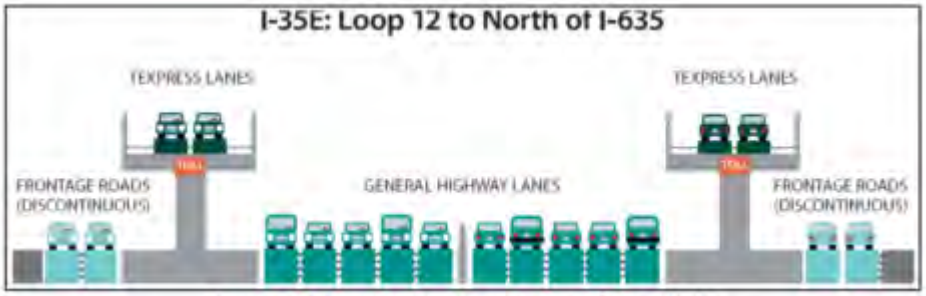

Figure 2. LBJ. Express toll connectors to/from IH 635

2. Managed lanes along IH 635. Considering the limited availability of Right of Way (ROW) along the corridor, a depressed U-section to accommodate the managed lanes had to be excavated and partially covered by the general purpose lanes, which were previously shifted outwards from their present position to create enough room in the median for excavation.

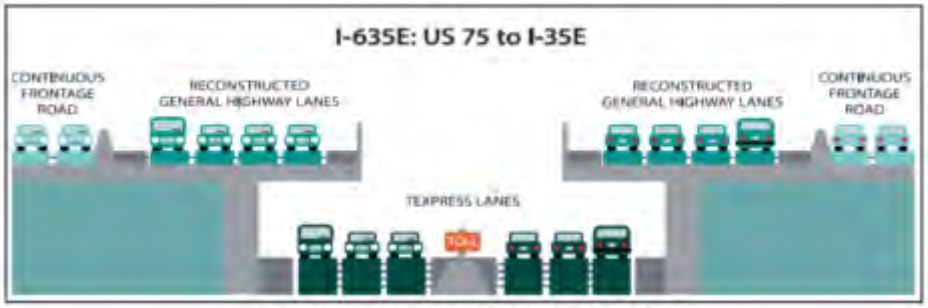

Figure 3. LBJ Express IH 635 Typical Section.

Several designs have been used in order to solve the specific problems presented by the determining features of this project described above. A description of the ones used for the IH 635 section is presented first, since it was the segment that required the most innovative design solutions, and afterwards the IH 35E direct connectors.

LBJ Express IH 635 section

The area of the Project with repetitive structural elements, which are most suitable for using precast elements as a viable option, is the depressed lanes where the general purpose lanes are partially covering some of the depressed managed lanes with the same structural pattern (see IH635 section in Figure 3).

The structures of the IH635 Section consist primarily of concrete beams with a length of 90-110 feet supported by bents caps spanning the managed lanes at widths of 60 to 90 feet in each direction of traffic.

The design and dimensions of the bent caps was a challenge since they had to be adequate for different construction phases, different span lengths, and at the same time their details and dimensions had to be similar for all of them in order to prefabricate them quickly, meeting the tight deadlines.

Several preliminary designs of the straddle bents for the U-section bridges were considered during the first stages of the design. Two primary alternatives were evaluated, the first consisted of precast

prestressed bent cap segments, and the second alternative utilized cast-in-place post-tensioned bent caps. Due to the magnitude of loads and the span lengths, a prestressed or post-tensioning solution was recommended.

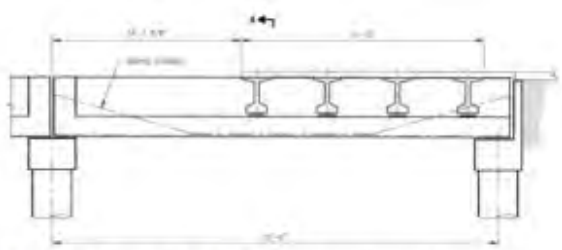

Figure 4: Alternative 1 – Precast/prestressed bent cap

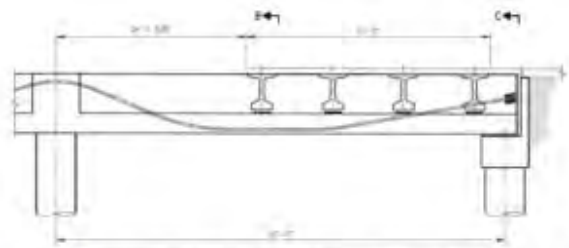

Figure 5: Alternative 2 – Cast-in-place post-tensioned bent cap

Alternative 1 was chosen for simply supported bent caps that could be precast in a plant, and installed in one night. Caps rest on two elastomeric bearings in each side, and column reinforcement does not have to be extended into the cap as it was typically done in the state of Texas.

An inverted-T section was chosen in order to reduce the vertical clearance, the shape of this section allowed a raise in the profile of the managed lanes, which reduces the height of the retaining walls, the excavation, and results in a more cost efficient design of the overall U-section.

All the bent caps had the same section: a total depth of cap of 7’-6” and width of bottom flange of 8’-0”.

The stem was typically 4’-0” wide, with a 2’-0” hollow in the center in order to reduce the weight of the cap and the amount of reinforcing steel and to make installation easier. The distance from the face of the stem to the center of bearing was 12” for TxGirders. The 2’-0” ledge width was found to be adequate for reinforcement to develop fully. The ledge depth varied along the bent cap in order to reduce the weight of the bent cap, 1”-6” was considered all along the cap, and 2’-6” was considered at the bearing seat locations in order to pass the bearing and punching shear check.

Bents caps were designed for HL-93 loading in accordance with AASHTO LRFD Bridge Specifications to be adequate for all construction stages. The inverted-T section had to resist all the combinations of service and factored loads required not only at the final stage but also in the intermediate construction stages where all beams and slab were in place in one side of the cap and no beams were installed in the other side. Flexure, shear and torsion were found to be adequate for all the combinations.

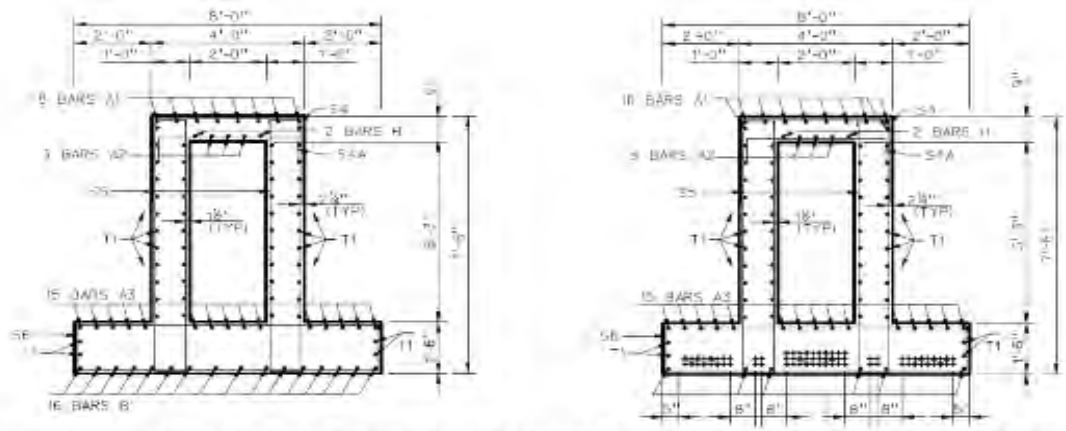

Figure 6: Inverted-T typical section (Precast with reinforced steel only, and precast with prestressed tendons)

There were several different type of bent caps depending on the span lengths and the construction phasing:

a) Precast design

1. Reinforced concrete

2. Pre-stressed concrete

3. Precast concrete pieces with post-tension

b) Cast in place design

a) Precast design

The standard type of cap used in Texas is a fixed cast-in-place cap. The time needed for the construction of a cast-in-place takes approximately two to three weeks per set of form, while in one night, five or six precast caps can be placed when the precast option is used.

There were three main designs depending on the geometrical and phasing constraints of the Project:

1. Precast with only reinforced steel:

These bent caps were mostly used when the span lengths were between 20 and 30ft, although some caps had spans around 45 feet where the reinforced steel was designed to handle the load, especially when the caps were supporting only one or two beams.

2. Precast cap with pre-stressed tendons:

The typical U-Section spans are around 60 feet, with variations from one cap to other. The precast plant was able to cast all the bent caps with only adjustments between one cap and other. With the precast caps, the construction team was able to place five or six caps per night.

Figure 7 shows a typical case which includes one cap with only reinforced concrete, and another cap with longer span where additional prestressed strands were required.

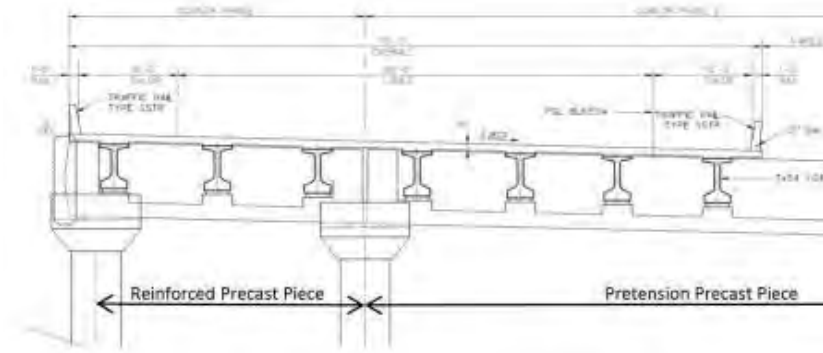

Figure 7. Precast caps with only reinforced vs precast and pre-stressed

Figure 8: Original situation, 4 lanes per direction on IH-635

Figure 9: Construction of exterior bridges

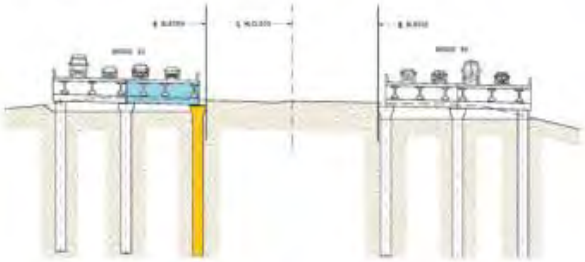

Figure 10: Second stage construction of exterior bridges

3. Precast concrete bent caps with post-tension

In order to maintain the amount of lanes to accommodate the heavy traffic, the bent caps were required to be constructed in several phases. See below for a description of the phases to explain the need of posttensioning between the different caps.

The original configuration had 4 lanes per direction in the IH-635.

Phase 1:

Temporary walls were constructed to maintain the traffic in the inner section while exterior bridges

were built. For this phase precast prestressed bent caps were sufficient to resist the traffic loads.

Phase 2:

At this stage, provisional drilled shafts that were acting as a temporary column shad to be built since a permanent column would be in conflict with the future traffic of the managed lanes as shown in Figure2. Prestressed bent caps with posttensioning ducts were installed and ready to be connected to the adjacent cap in a later phase.

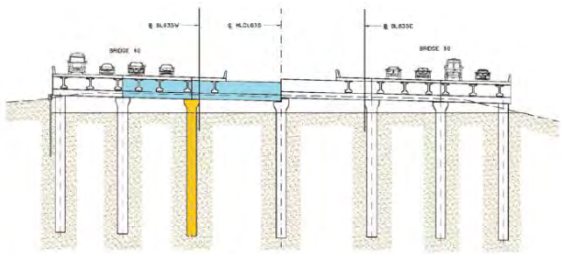

Figure 11: Bridge Completion

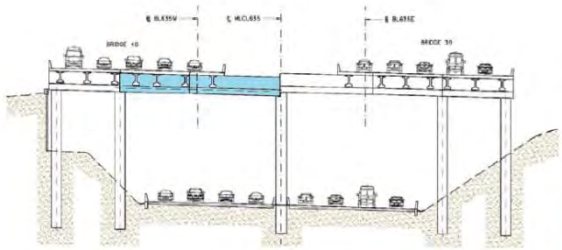

Figure 12: Excavation and Managed lane construction

Phase 3

Another cap spanning in most of the cases more than 60 feet was placed between the existing bent caps and the center pier of the managed lanes. Posttensioning of bars and tendons was required to connect the bent cap with to the previous cap. Details are explained later in the article.

Phase 4

Once both bents were connected and post-tensioned, the provisional drilled shaft was removed, the beams could be placed, and the rest of the deck poured. The final step consisted in the excavation and the construction of the managed lanes. Face of drilled shafts were cleaned and aesthetics were included in the columns.

Post-tensioning caps were carefully designed since the amount of reinforcement had to comply with the minimum reinforcing spacing so the concrete could be poured easily around the reinforcement. Three dimensional drawings with real dimensions were needed to make sure that the upper tendons, lower tendons and reinforcement were not interfering.

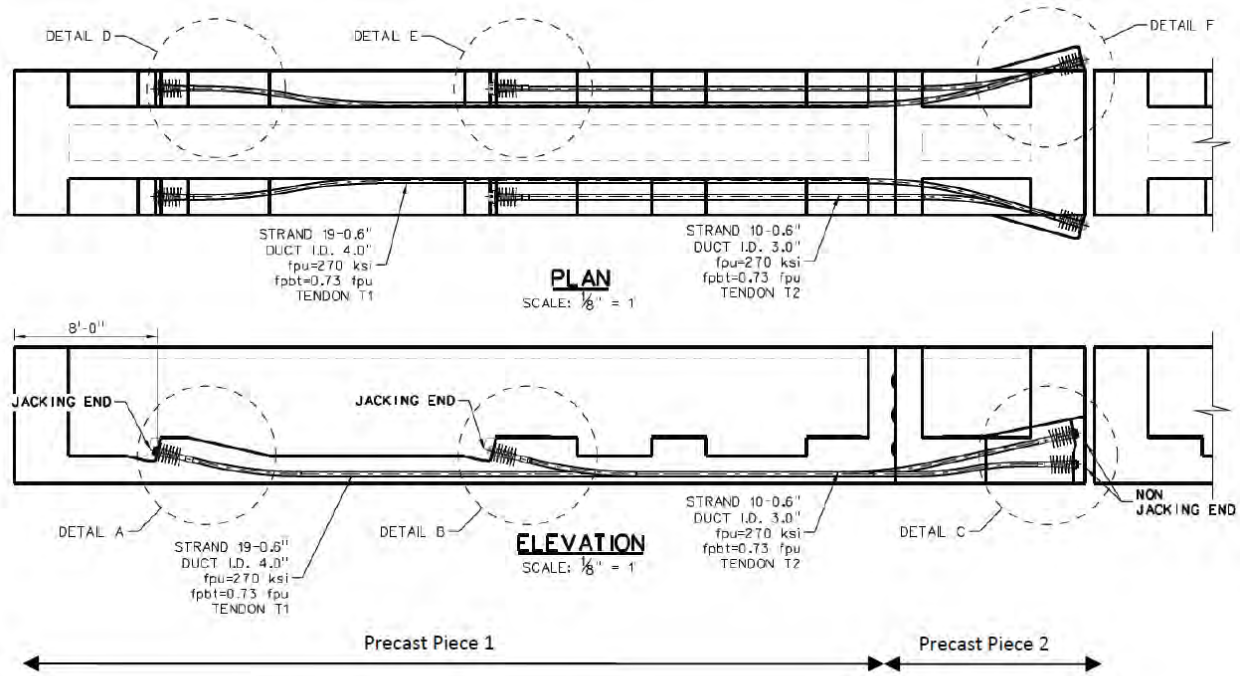

Figure 13: Bent general details with post-tensioning tendons.

Once both caps were in place and the concrete strength reached 6,300 psi, 10x1.75” prestressing bars connecting both caps were stressed with a typical jack force (before anchorage) of 315 kips. After bars were stressed, two different families of tendons had to be stressed as well. For a typical 50ft bent cap, the first 2 tendons (T1) with 19x0.6” strands each inside a 4” duct was stressed with a jack force (before anchorage) of 810 kips. The second 2 tendons (T2) with 10x0.6” strands each inside a 3” duct was stressed with a jack force (before anchorage) of 430 kips. The number of strands and jack force varied slightly depending on the span lengths. Details in the anchorage zone were carefully designed to resist the post-tensioning forces.

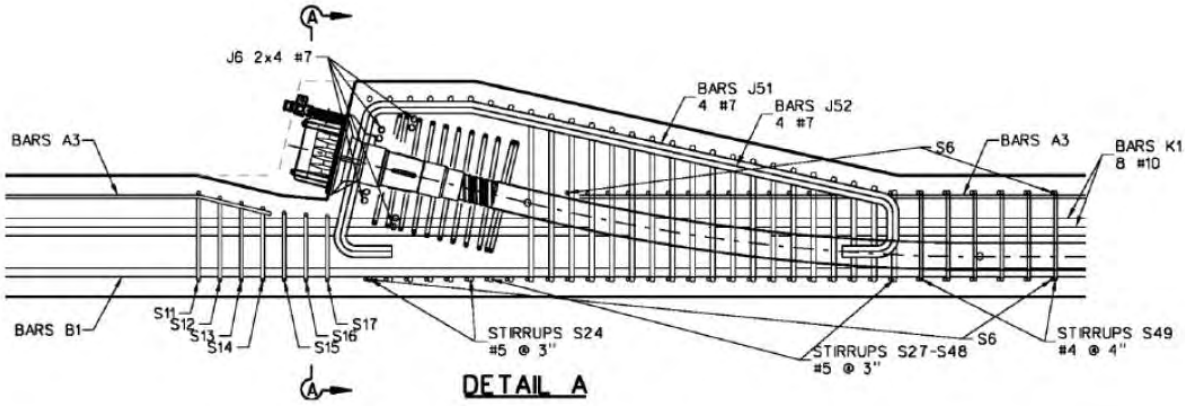

Figure 14: Jacking end. Anchorage zone details.

Figure 15: (a) Post-tensioning bent cap; (b) Precast Bent caps.

b) Cast in place design

Cast in place design was considered in 27 bent sections out of 217, with a total of 86 cast-in-place portions of bent caps versus 491 of precast bent caps. The span length of the cast-in place caps vary from 50 to 70ft. No prestressing was needed since the cap was designed continuous along the section. In order to achieve this continuity, mechanical couplers were installed at the construction joints between caps so the reinforcement was continuous. For very few bent caps (less than 20) with longer spans, up to 90 feet, posttensioning was needed due to this increase of length.

To describe the importance of this innovation, Figure 16 shows one of the few original fixed cast-in-place caps constructed. The use of cast-in-place caps as shown in Figure 16 would have not only increased the time needed to finish the project, but also would have increased the potential risks for accidents due to employees working at great heights.

Figure 16: (a) Reinforcement placement in cast-in-place cap; (b) Reinforcing cage; (c) Concrete pouring in cast-in-place cap; (d) Cast-in-place cap finished.

LBJ EXPRESS IH 35 DIRECT CONNECTORS

Composite bent caps IH 35

IH 35 direct connectors were elevated structures along the corridor which had to span over the existing

IH-35 in some cases originated a construction conflict that had to be solve in an efficient manner and minimizing disruption to existing traffic.

As part of the construction of the elevated managed lanes crossing over the existing IH 35 south bound, the preliminary design first utilized conventional cast-in-place caps. That solution required the a full reconstruction of a portion of the highway through the existing frontage roads as a temporary detour of 3 lanes of heavy interstate traffic. To avoid this situation, a composite structure was proposed which would avoid the construction of the detour and provide a significant reduction in construction time.

Figure 17: Location of the composite caps in order to avoid a massive traffic deviation.

Composite caps were placed overnight, causing only a night closure. These composite caps were formed with steel caps with a concrete slab on top to account for the compressive stresses. Finally, after placing the girders, the precast concrete panels and pouring the bridge slab, the bridge was finalized. The use of this composite cap allowed not only to avoid construction of a massive traffic detour (that would not be used in the future), but also it allowed to overlap other activities that were needed in the location where the detour was going to take place.

Figure 18: Composite cap section, and cap installation

CONCLUSIONS

When facing a tight schedule with complex traffic phasing and space restrictions innovative ideas are needed to succeed under these challenges.

These innovative ideas have to be adjusted to each specific problem encountered in the project. It has been shown how well precast elements have worked when similar designs allowed to use a standard model. These precast elements had to be adjusted to the construction phasing and post-tension became the solution to solve some of the most difficult phases of the construction process. In addition, in order to solve a different type of problem, a composite section helped to minimize traffic disruption to a very heavily trafficked highway.

Precasting has been one of the keys to the success of this projects and ought to have an special consideration for this section. Some of the main advantages of using this method are listed below:

- Schedule improvement: in addition to the time savings when precasting in a yard on site versus cast in place, construction of the caps could begin prior to excavation which allows several other activities to be overlapped.

- Ability to adapt the structures to the traffic control plan: some bridges were built in differen phases. In order to connect the bridge, as a temporary solution, the caps were placed on temporary columns with temporary seats and then it was assembled with the rest of the caps with post-tensioning.

- Construction safety is improved: precast caps are built in a controlled facility. Conditions on site such as heights, heavy machinery, traffic, etc., impose more hazards and risks to the workers.

- Improved Quality: At the facility, the workers have a routine, they have specific tasks and it is also easier for quality inspectors to perform their jobs. Additionally the forms in the plant were provided with external vibration which meant a better finish product.

- Easier to transport and place: precast caps were less heavy than the cast in place option and could be easily transported and place in their final position.

And finally, common practice is the use of cast-in-place caps, but for the LBJ project alternative designs such us precast, post-tensioning and composite caps were implemented in order to meet all the needs and requirements of this project. By combining the different options described in this document, the final

design was able to accommodate a traffic control plan needed for the construction and to reduce the timing needed to finish the bridge structures along the corridor. The use of a composite solution avoided a massive traffic detour, saving time and increasing safety during construction.

5 ACKNOWLEDGEMENTS

The authors wish to express their respect and admiration for TxDOT and their leadership in the project described above. They also want to acknowledge the General Engineering Consultant working for TxDOT Jacobs and the Independent Engineer URS for their participation in this proect.

This process of incorporating innovative design ideas for their inclusion in the final product was possible with the collaboration between all parties which lead to a successful delivery of the project ahead of schedule. The technical descriptions that appear in this paper are from the authors' viewpoint as a committed but private proposer and are not intended to represent TxDOT's position.

BRIDGE NO. 465 REPLACEMENT, I-195 RAMP (DR-2) OVER WARREN AVENUE, EAST PROVIDENCE, RHODE ISLAND

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INTRODUCTION

Warren Avenue Bridge No. 465 is the on-ramp to Interstate 195 westbound, which carries Veterans Memorial Boulevard over Warren Avenue in East Providence, Rhode Island. The proposed bridge design implemented a variety of Accelerated Bridge Construction (ABC) techniques, which in their totality were new to RIDOT, such as Precast Concrete Elements (PCEs), Prefabricated Superstructure Modular Units (SMUs), the use of crushed stone for foundation support and backfill, and Systems and Structure Erection Techniques. Combining the various techniques significantly lessened the impact on traffic and the public and significantly reduced bridge closure time, from 11 months to 22 days.

The existing structure, constructed in 1959, was a three-span, simply supported bridge, with an overall length of approximately 123 feet. The existing roadway width measured 23 feet and carried one lane of traffic. The existing superstructure consisted of prestressed concrete beams for the center span and reinforced concrete beams for the two end spans. The existing substructures consisted of reinforced concrete piers and abutments supported on piles. The existing piers were shored with timber cribbing due to their advanced stage of deterioration.This structure was posted for 10 tons.

The narrow existing roadway width and girder layout eliminated the option of staged construction. Therefore, the bridge replacement required the complete shutdown of the I-195 on-ramp (DR-2). Detour routes were identified to provide travelers with a connection to I-195 westbound while the ramp was

closed, as well as eastbound/westbound access for two weekends of closure of Warren Avenue. At the beginning of the design process, the project team used the Federal Highway Administration's (FHWA) Framework for Decision Making for Prefabrication as the basis for recommending the use of ABC systems for this project. In fact, this project received implementation assistance for Innovative Bridge Designs for Rapid Renewal (RO4) and was showcased as part of FHWA's second Strategic Highway Research Program (SHRP2) on October 30, 2014 in Providence, Rhode Island. The showcase presentation of the bridge can be found on AASHTO's SHRP2SOLUTIONS web page at http://shrp2.transportation.org/Pages/Bridge-Designs-for-Rapid-Renewal.aspx

FOUNDATION TECHNIQUES

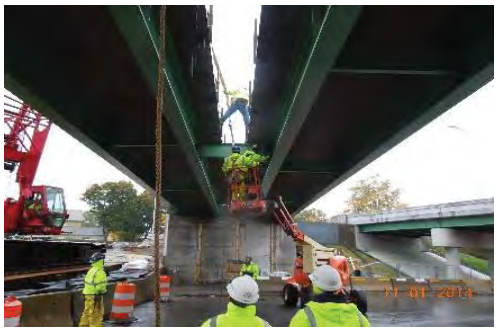
The geotechnical ABC techniques included the use of reinforced crushed stone to support the foundation and approaches. The reinforced crushed stone provided a much needed improved bearing capacity, thus eliminating the need for pile supported foundation that would have required additional time to install. The reinforcement consists of structural geogrid that is placed at every one-foot of compacted crushed stone depth. Additionally, the use of crushed stone backfill eliminated the constraints of the typical gravel borrow fill and required less compaction time. A significant amount of rain fell on the site during the first week and the use of crushed stone backfill prevented any delays.

The new bridge abutments and wingwalls consist of precast stems and footings that are founded on the reinforced crushed stone. The precast stem footings are skewed to accommodate the 25 degree bridge skew. The precast wingwall footings were stepped to avoid deep open cut foundations, reduce earthwork, and to reduce the need for Support of Excavation (SOE).

COMPLEXITY

The angles of the horizontal roadway alignment for Ramp DR-2 and Warren Avenue require the new bridge to be constructed with a severe skew of 25 degrees. Conventional wisdom would dictate that prefabricated bridge components be constructed primarily of rectangular shaped units at right angles. To overcome this constraint, special details were developed for connecting the return wingwall stems to the abutment footing units.

For this bridge site, a SOE system was required to protect the adjacent existing bridge structure, street, and trees. To mitigate the need for a much more extensive SOE, the project team was able to step the precast wingwall footings due to the support achieved from the dense compaction of the reinforced crushed stone beneath the footings. Stepping the precast wingwall foundations required considerably less earth removal and SOE. Additional time savings were realized by detailing the footing steps to be prefabricated on top of supporting footing. To ensure that there would be no delays due to rain while backfilling behind the stem walls, crushed stone was also used as fill behind walls.



A single-span, steel multi-beam superstructure spanning 84 feet was selected to replace the existing structure. The two prefabricated SMUs consisted of a reinforced concrete deck with two rolled steel be ams. The SMUs were prefabricated onsite in a staging area near the existing bridge. They also included safety barriers, which were cast onto the modular units before each unit was lifted in place. Once both SMUs were in place, they were connected by a 2-foot-4-inch wide closure pour.

The severe skew also affected the constructability of the prefabricated SMUs. To stay within acceptable tolerance limitations between the adjacent SMU units, the specification was written to require the two SMUs to be prefabricated and poured as a single bridge deck with no concrete in the closure pour strips. This requirement also eliminated the potential for misalignment of the SMU's when placed on the beam seats.

ABC SUCCESS

Louis Berger designed and detailed this bridge to be constructed in 30 days or less, with two weekend closures of Warren Avenue traffic in the impacted area near the bridge. The contractor's 30 day window counted from the demolition of the existing structure to the reopening of Ramp DR-2. The first weekend closure marked the beginning of the 30 day period and was used for the demolition of the existing structure. This first weekend closure occurred successfully on October 18th through October 19th. The contractor then had two weeks to complete the excavation and construction of the new substructure. The second weekend closure was scheduled to occur halfway through the 30 day period, which was used to place the SMUs. This event successfully occurred on October 31st through November 2nd, and only three hours to place the SMU's. The contractor achieved substantial completion and reopened Ramp DR2 on the 22nd day after the bridge closure, eight days ahead of the 30 day schedule.

FUTURE VALUE TO THE ENGINEERING PROFESSION

For this project, Louis Berger demonstrated how PCEs and SMUs could be utilized for more than just "ideal bridge sites" suited for conventional precast concrete construction. The particulars of this bridge site and time constraints for construction would normally have invalidated the use of precast concrete for this project.

The precast manufacturer for this project was challenged to go beyond industry norms to show its ability to compete with other emerging technologies. Engineers considering the use of precast construction on difficult sites may learn from this project. This project also demonstrates how the combined use of various ABC techniques were used to overcome a difficult construction site and poor weather conditions.

PERCEPTION BY THE PUBLIC

Public reaction and interest regarding the project has been positive. Throughout the design process, Louis Berger and RIDOT met with the city engineer planner to gain its approval and support, held many meetings with the city's police department to ensure that they understood the project scope, and presented the project to the City Council and to the public The project team's approach was to explain ABC construction techniques as "Lego pieces", which best explained the concept and value of the techniques to the community. Louis Berger also presented its 3-D model video to the public to further assist in visualizing the project components. An animated video of the bridge construction is available on YouTube at https://www.youtube.com/watch?v=0K0OElDUC10&feature=youtu.be. Subsequently, the local community approved the ABC construction approach and supported the bridge replacement.

SOCIAL, ECONOMIC AND SUSTAINABLE DEVELOPMENT

The ABC techniques employed for this project significantly reduced the total time of construction and, as a result, significantly reduced the overall disruption to the environment. Furthermore, the use of prefabricated precast concrete elements, made in a controlled plant environment, significantly reduced the onsite construction debris, waste, and potential for contaminated runoff. Eventually, after Bridge 465 has served its useful life, the concrete and steel can be recycled for other uses.

SUCCESSFUL FULFILLMENT OF OWNER NEEDS

There were significant time savings as a result of using ABC techniques over conventional construction methods. RIDOT completed a similar-sized bridge (#464) adjacent to this bridge using conventional construction for superstructure replacement and substructure modification only. The closure of Bridge 464 (from demolition to traffic opening) was 10 months. Estimates indicate that the full superstructure and substructure replacement of Bridge 465 using conventional construction would have taken approximately 12 months. The closure of Bridge 465 from demolition to traffic opening only required 22 days, triggering a contract provision allowing the contractor to collect incentives while significantly reducing the traffic impacts on the public.

SELLWOOD BRIDGE DIVERSION ALIGNMENT AND BRIDGE TRANSLATION

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Multnomah County's Sellwood Bridge is a member of the iconic collection of Bridges crossing the Willamette River in Portland, and is the busiest two lane bridge in the state of Oregon, serving over 30,000 vehicles per day. The circa 1925 bridge has passed its useful life and is currently being replaced with a steel deck arch. To avoid the added cost of a detour bridge across the river, scheduling issues and difficulties of staged construction for a two rib deck arch, this project utilized the unique opportunity to the 88-year-old bridge main span truss as a temporary detour structure. The existing spans were translated north 33 feet at the east end and 66 feet at the west end onto temporary supports. Accomplished during a one week closure of the bridge, this approach was estimated to have saved the owner approximately $10M in construction costs.

The approach to designing a temporary structure differs from that of a permanent bridge. The engineer must consider the approach to both loading and resistance of structural members in the context of a temporary service life. The design of the new support system was complicated by the fact that the 3,200-ton, 1,100-foot long truss is continuous, requiring that translation be accomplished as a single piece with the detour alignment requiring that the structure be skewed. The owner is utilizing a CMGC contract for this project, allowing for close collaboration and coordination between the owner, the engineer, and the contractor.

Slide in Bridge Construction (SIBC) Procurement

In the CMGC Process the selection of the jacking and SIBC sub-contractor was pursued in an open forum with the CMGC, the County, the Engineer, and the competitive sub-contractors participating in a work session. Two SIBC sub-contractors were considered, each presenting a unique approach to the SIBC in the context of the contract procurement documents.

The first subcontractor approached the project with a concept of jacking similar to that envisioned by the Engineer. The basic steps to the process being:

1. Set up jacking system around the existing bearing locations

2. Lift the bridge and install remaining skid track under bearings
3. Slide the strong-back skid into positon under the bearing locations
4. Translate the bridge on skid surfaces along a single line per structure support location
5. Lift the bridge from the strong-back and remove the skid system
6. Lower the structure onto the temporary support bearing locations

The second subcontractor provided and approach where the skidding system was installed outboard of the bearing locations and through the addition of additional structure the reaction force was transferred from the bearing line to the outboard skids. The sequence consisted of:

1. Set up of jacking system around bearing locations
2. Lift the structure
3. Translate along the skid lines
4. Lower onto temporary bearing locations

As the SIBC concept is inherently time sensitive the second approach was favored as providing fewer steps to complete the construction and requiring a minimum of construction processes during the road closure. The second system was also deemed to be more economical given a lower price quote from that subcontractor, leading to selection of that system

This system did have other inherent expenses, however. The system required a significantly larger width for support f 2 skid lines per bearing location as opposed to one. This required re-design of the temporary support towers and consideration of loading along the two skid alignments. This issue was complicated given the basic requirement to re-align the structure. The off tracking of the skid alignment on a curve introduced torsional moments to the chorded supporting beam members. This was addressed through the use of substantial cross bracing and relatively heavy cross ties at the supporting deck level.

To offset this additional steel weight required the temporary tower sections were re-designed further to an “A” frame configuration as opposed to vertical legged tower. This was opportunistic given that the initial design reflected a desire by the CMGC to incorporate that tower as temporary support for the structure during demolition, i.e. configured to support the structure in various pieces as it was demolished. These plans had not come to fruition and the concept was abandoned, allowing for redesign to accept the lower cost skidding system without significantly increasing the volume of steel constr4uction required to support the reconfigured SIBC installation.

SIBC Progress, The Day of

The SIBC portion of the project was planned for roadway closure over the course of a weekend. Weather was cold, below freezing with fog materializing through the evening hours and clearing in the morning with no appreciable wind velocity.

The weather situation is significant. The timing of the SIBC, in January, increased the potential for inclement conditions, which did not materialize.

Control of SIBC slide processes were from the centrally mounted hydraulic pump unit. From this unit vertical jacking, and control of the pushing tug hydraulic rams were operated. Each location carried a crew to monitor the progress and report to the central control at each movement of the structure. The control to move the structure into the modified alignment was achieved by varying the pressure to each location according to the mathematical balance between each bearing location. This achieved the desire result of the west end moving 2 times further than the east end of the structure. No specific guide was used to steer the structure, the variable hydraulic pressures achieved the turn along the required arc alignment.

At each incremental push the structure progress was measured and reported. Throughout the process various monitoring devices had been installed to ensure that the structure was not unduly stressed. These included:

- Strain gages mounted along the structure with real time monitoring
- Ground survey with laser distance measure to various points along the structure
- Laser line measure within the structure to ensure consistent alignment
- Marking an all skid tracks

Most electronic measurements proved ineffective at real time monitoring as the vibration that ensued during movement produced highly variable scatter in the collected data which proved to be unreadable. At each pause to reset, however, the data settled and could still be used to monitor the progress and observe any issues indicating potential difficulties. None were observed throughout the process.

Monitoring in Service

The diversion structure continues to be monitored in its current location. Scour depths are monitored continuous via electronic sonar methods and the structure receives a visual inspection annually. Flood waters resulting in scour being the primary threat to potential structure damage. To date no such extreme weather events have been observed at the site to warrant any special inspection. The structure is currently

performing in its third, and final, winter season and has provided service without notable incident or need of any repair activities.

Lessons Learned

The CMGC procurement proved highly valuable to the ability to select an effective SIBC subcontractor. The procurement, however, did present some difficulties. IN this case the procurement. or bid, drawing depicted a structure configuration that, upon selection of the SIBC sub-contractor, required significant re-design to accommodate the system. This resulted in a change of information to the steel fabricator and some additional costs being incurred in that regard. Essentially the selection of the low price from the SIBC subcontractor was concluded without fully appreciating other unintended costs that were later incurred. The lesson being that diligence is needed, particularly when the concept presented differs from the basic configuration of the bid documents.

Installation on a modified alignment is viable and achievable. The low friction skid surface is sufficient to slide the structure in all directions and achieve the desired result. This was not immediately apparent during the initial design. The construction did assume a basic 5% lateral loading during the skid process, and this proved to be sufficient.

Structures placed into temporary service require greater definition in the national design codes. Seismic requirement recognize the probabilistic nature of the event in comparison to the length of service. Service of 5 years or less allowing for a 50% reduction in the demands applied. Other probabilistic demands, such as wind, flood (scour) and vessel collision do not similarly provide for codified reductions against a structure in temporary service.

SIBC is highly recommended for any environment where traffic impacts should be minimized. In this case a highly urbanized environment with limited right-of way.

The move was well documented, with many photos and videos. The presentation will conclude with some lessons learned.

RECONSTRUCTION OF THE WILLIS AVENUE BRIDGE

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ABSTRACT

The main span of the new Willis Ave. Bridge is a 350' steel truss swing bridge connecting Manhattan and Bronx. It is a major highway over navigable water in the heart of New York City, carrying over 62,000 vehicles a day. The new bridge was prefabricated and pre-assembled in Albany, NY. The bridge was towed to Manhattan on barges and it was floated onto the center pivot piers on two barges in a catamaran setting.

The ABC techniques use in these projects effectively minimized the impact on traveling public. The proper implementation of ABC yielded significant savings on the construction cost.

INTRODUCTION

In the late 1890's, famous bridge engineer Thomas C. Clarke was selected as the engineer of record to design the then-new Willis Avenue Bridge. The new bridge would replace an existing iron bridge that was built in 1868 with only a clearance of five feet above high tide. Mr. Clarke's new Willis Avenue Bridge has a 304-foot swing span, flanked by plate girder spans to the west and a 240-ft through truss span to the east.

The existing Swing Span has a rim bearing with distribution girders. When open to marine traffic, the Swing Span provides two channels each with a clear width of 112 feet, and unlimited vertical clearance. In the closed position, the swing span allows 24 feet of vertical clearance above Mean High Water. The use of the Swing Span can greatly lower the overall roadway profile and eliminate the need for elevated viaduct structures in the busy Manhattan streets.

The mainline structure carries four lanes of traffic over the Harlem River Railroad Yard (HRY) and serves as an approach to Willis Avenue in The Bronx. The Harlem River Yard is under development as an intermodal transfer facility and industrial/commercial park. There are currently six railroad tracks under the bridge in this area. Four the tracks serve a Waste Management municipal waste transfer facility, one is the CSXT Railroad Oak Point Link and one is intended to serve an intermodal facility. One additional

track is planned at the intermodal facility. The bridge also spans over access roads within the rail yard, several city streets, and a bus turnaround loop connected to the 126th Street bus Depot in Manhattan.

The Willis Avenue Bridge is a principal northbound route between the Boroughs of Manhattan and The Bronx in the city of New York. In addition to carrying four lanes of traffic across the Harlem River the Willis

Avenue bridge is a key link to the Interstate Highway System. Like the six other swing spans and four lift spans over the Harlem River, the Willis Avenue Bridge Swing Span allows most vessels to pass without opening but opens periodically to permit the passage of tall vessels.

Figure 1 – Willis Ave. Bridge (built in 1900's) open for navigation

After 100 years of service, the bridge structure was desperately in need of a complete overhaul. By the early 2000s, the Willis Avenue Bridge exhibited the effects of age, weather and continuous daily usage by motor vehicles. As part of a $612 million project the NYCDOT completely replaced the bridge, including the FDR Drive approach ramp and the ramp onto Bruckner Boulevard in 2010. The project is a major component of DOT's long range Harlem River Bridges program, which has so far rehabilitated or replaced the Macombs Dam, Third Avenue, Madison Avenue, 145th Street and University Heights Bridges.

This work is part of more than $5 billion in bridge investments made by the Bloomberg Administration since 2002, which includes rehabilitation and repainting projects on all of the City-owned East River bridges.

Figure 2. Willis Ave Bridge prior to demolition (145th Street Bridge shown on barge in the background)

CONSTRUCTION MEANS AND METHODS

In 2005, the General Contractor, Kiewit Construction and Weeks Marine, JV, the successful low bidder of the project, signed the contract with the City of New York to execute the project. As a common practice in

US the general contractor is responsible to select the means of methods of construction. It is also the responsibility of the general contractor to retain his/her own Construction Engineer to prepare all engineering design and calculation for implementing the planned construction means and methods.

Prior to the bid, UrbanTech entered an agreement with Kiewit-Weeks, JV to provide pre-bid engineering service. Should the team led by Kiewit-Weeks, JV become the successful bidder, UrbanTech would provide all engineering services related to the demolition and erection of swing span of the new Willis Avenue Bridge. This relationship is similar to typical contractor-engineer arrangements in a design-build contract.

After numerous brainstorm sessions, it become apparent to the team that the most effective method of construction for the swing span was to fabricate and erect the entire swing span off-site while the team in

New York is constructing the approach spans and bridge piers. Upon completion of the fabrication and assembly, the new swing span will be delivered to the job site in Manhattan via barges. Final setting of the fully assembled swing truss span will utilize the “float-in” method.

Due to the final configuration of the bridge design, the swing span must be placed on two barges in a catamaran setting in order to clear the center pivot pier. At high-tide, the swing span will be eased into position by tugboats and guided by several winches after which the swing span will be set onto a temporary jacking and positioning system for initial setting. Float-in barges will be removed to clear the

navigation channels. The swing span truss will be lowered on the permanent spherical center bearing with high precision hydraulic jacking-skidding system.

The reconstruction of the Willis Avenue Bridge is an example of textbook implementation of accelerated bridge construction methods. The use of ABC allowed the general contractor to compress the construction duration and reduce the impact on the local traffic. In fact, the benefit of minimizing the impact on local traffic is just a by-product of other more immediate needs for implementing the ABC technology for this project - better quality, lower cost and shorter duration.

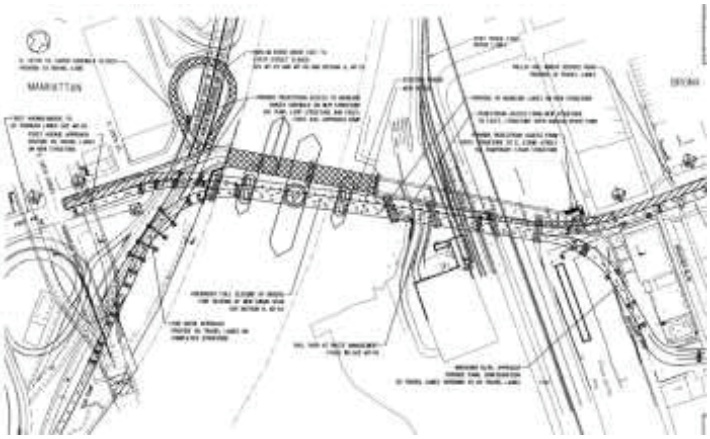

Figure 3. Demolition of the Existing Bridge after completion of the new bridge

The new swing span of the Willis Avenue Bridge has a spherical bearing at mid-span with balance wheels that are designed to take minimum unbalance load due to incidental eccentricity and wind load. When the swing span is closed for vehicular traffic, the span ends are engaged, or lifted, by a set of span-end lifting devices, forming a two span continuous condition for live loads. The span-end lifting devices is designed to lift the span-end up by 25mm with 1.2 mm tolerance. The geometry control program of the main span is designed to control the final profile of the swing span within the desirable 1.2mm tolerance. Considering the total span length of truss (350 ft or106m), the 1.2mm tolerance is very stringent.

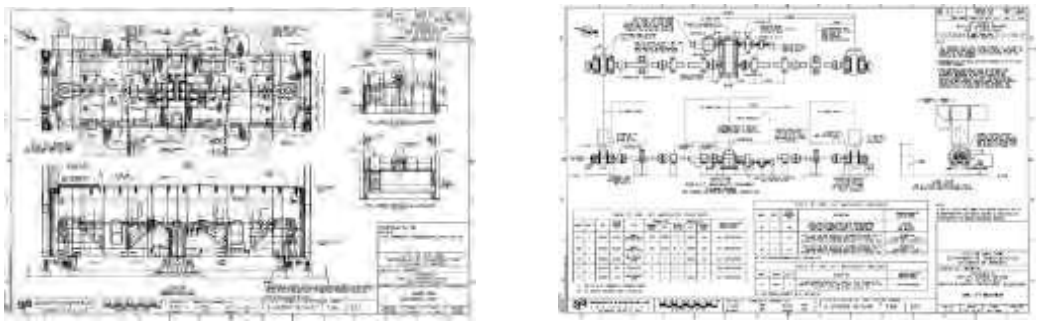

Figure 5. Complexity of the machinery

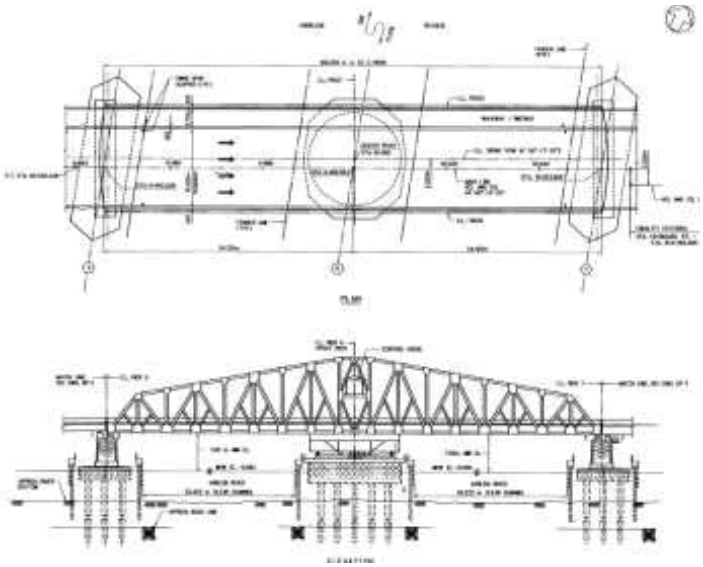

Figure 6. Plan and Elevation of the Willis Ave. Bridge

The contract specification also requires that the truss fabrications and erection procedure to be designed such that the secondary bending stresses in truss members due to dead load are to be cancelled by means of pre-stressing. This procedure greatly reduces the local stresses near gusset plates of main connections and improves the overall fatigue performance and the durability of the bridge structure. It also allows for the designer to reduce the self-weight of the bridge superstructure. This procedure, however, requires high precision fabrication and erection. The fabrication tolerance for member length is less than 0.5mm for all truss members. The erection must have the same order of precision. On-site erection of the truss would require the installation of an extensive falsework system over the navigation channel, which makes this option practically impossible.

Figure 7. Bridge on Falsework

The construction team selected a bridge erection site located in upstate New York on the West bank of the Hudson River at a port known as Port of Coeymans. All structural components were trucked and/or barged to the erection site from the steel fabrication shop in Pennsylvania and Alabama. Erection falsework was set to the proper elevations to accommodate transportation and final float-in operations. The falsework included hydraulic jacks that were installed to allow for adjustment of vertical profile of the bridge truss.

The completed bridge was transferred onto a twin-barge system using SPMT (self-propelled modular transporter). The transfer of the bridge from land to water represented a challenging engineering task. It requires comprehensive engineering analysis and careful planning to count for trimming, listing of the barge, and river tidal change.

Figure 8. Load-out

The operation started at low tide and continued throughout the entire tidal cycle. High capacity pumps were used to control the free board, trimming and list of the barge during the entire load-out operation.

The bridge truss were kept level during the load-out operation.

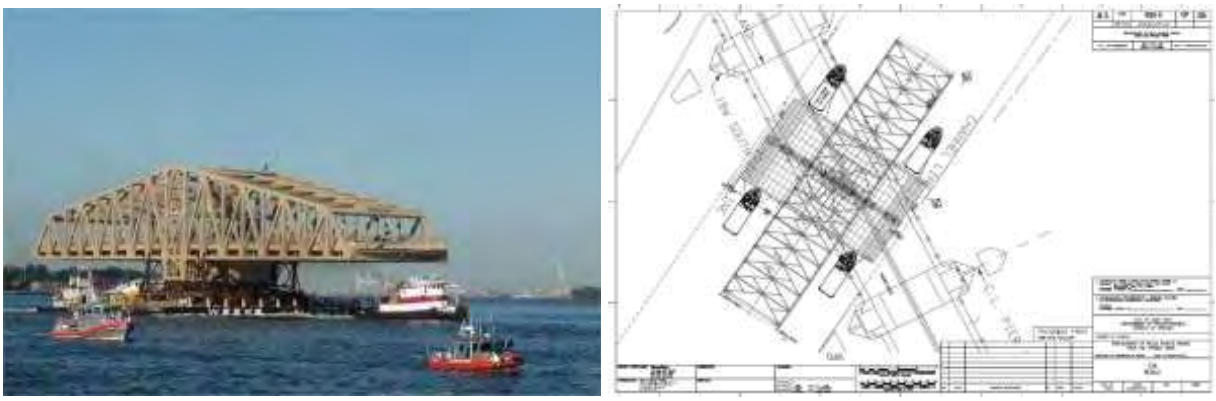

Figure 9. Bridge being towed to Project site in Manhattan

Upon completion of the load-out, the transport barge was towed to the project site in Manhattan. Due to the configuration of the bridge superstructure and substructure, the bridge must be transferred from the transport barge onto a pair of “float-in” barges. The float-in barges and bridge superstructure are set in a catamaran configuration to clear the center pivot pier.

Figure 10. Bridge being transferred onto “float-in” barges

Figure 11. Construction of the Sub-Structure

The adoption of the accelerated bridge construction approach greatly improved the construction schedule. The duration of on-site construction activity related to the super structure of the main span was compressed to two months. The erection of the swing span was completed under a well-controlled environment with proper equipment and the contractor delivered high quality product.

ENGINEERING CHALLENGES

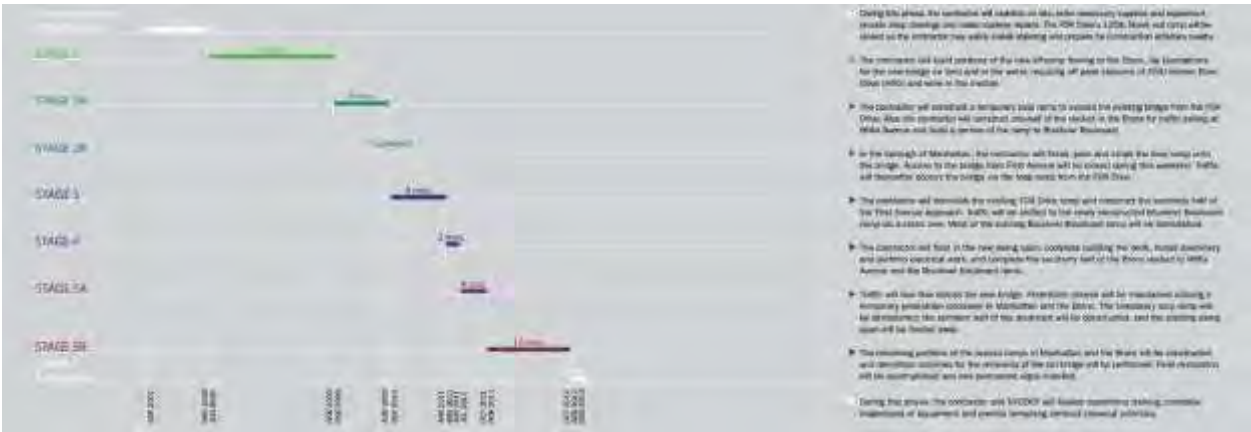

Figure 12. Staged Construction and Sequence

The implementation of ABC technology presented numerous engineering challenges to the construction team. Many aspects of the project cannot be clearly categorized into conventional bridge engineering practice. Design standards and specifications were not readily available. The engineering teams and construction management staff were constantly struggling to come to an agreement on which codes and regulations are applicable to certain design tasks. More often than not, the design team had to go back to the engineering fundamentals to search for answers, rather than following standard codes and regulations. The best codes that were applicable to this type of work was to apply basic theory of elasticity with the help of powerful finite element analysis tools. Applicable standards and specifications included the following:

1. AASHTO Specifications for Highway Bridges (ASD or LRFD)

2. AISC Manual for Steel Construction

3. Standard specifications for Bridge Construction (State DOT's and AASHTO)

4. DNV Rules for Planning and Execution of Marine Operations

Structural and Marine engineering issues to be addressed during the design and planning stage shall include the following:

1. Constructability

2. Strength and stability of bridge-vessels system

3. Effects of the transportation method on the fatigue service life to of the bridge structure

Constructability

The selection of the barges are based on the availability of the marine contractors in the region. For the transportation of the bridge, UrbanTech proposed to use a twin barge system that are made of two identical barges, connected side-by-side. The port side of the first barge is connected to the starboard side of the second barge. The bridge was loaded transversely onto the barge, forming an unusual orthogonal configuration during the tow. The center of pivot of the swing span is supported at mid-ship of the twin barge system with the span ends projecting beyond the footprint of the transport barge. This configuration appears to be more vulnerable than conventional setting at first sight, however, detailed analysis indicates that this configuration provides better dynamic characteristics and system stability.

The key advantage of this unconventional setting is to allow easy transfer of the bridge structure from the tow barge onto the “float-in” catamaran barges (see Figure 10). This is just one example of how constructability was carefully considered and it truly became the driving force behind all of the engineering and planning processes of the project.

Strength and Stability bridge-vessel system

The erection facility for Willis Avenue Bridge, Port of Coeymans, is about 120miles north to the project site in Manhattan. The inland river tow from Coeymans to Manhattan project site takes about 2-3 days at the design speed of 5 knots. The expected environmental loads were minimal. The lashing system was designed for impact load in case of accidental collision and damage to one compartment of the barge. The effect of impact load on the barge-bridge system was carefully analyzed using Finite Element Method.

In addition to the impact load, dynamic effects induced by wind loads were also analyzed using Finite Element Analysis method. Maximum anticipated acceleration of the system was analyzed based on the frequency of the system and the maximum anticipated equivalent static design wind load. Static loads due to trimming, listing also need to be considered when designing such a system.

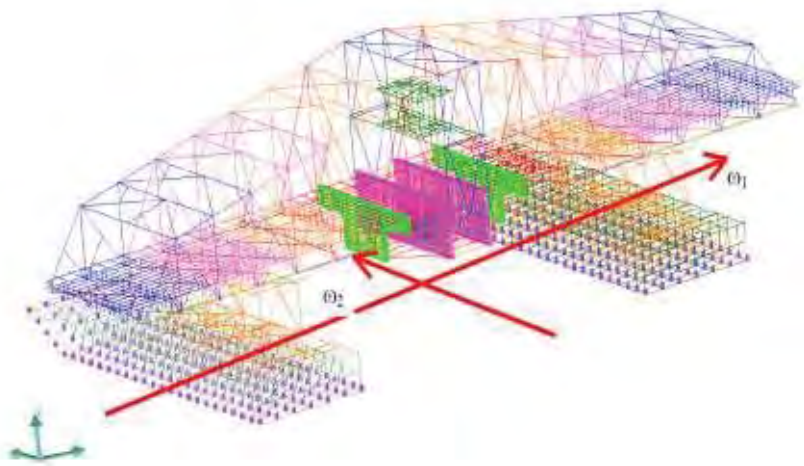

Figure 13. FE Model of the Float-in Catamaran system

Stability of Catamaran system

When placing bridge structure on barges or other type of vessels, the overall stability must be studied carefully using the principles of naval architecture. Generally speaking, the stability of a catamaran system is significantly better than a single haul vessel. However, the stability of a rudimental catamaran system, such as placing a bridge on top of two barges, without a dedicated connection structure may not be able to work compositely to form a true catamaran.

When the bridge is supported by two barges in a catamaran setting, the stability of this system needs to be studied. The metacenter of the combined system can be found as a function of the following:

1. Hydrostatic properties of each individual barge
2. Stiffness of bridge structure and supporting falsework
3. Spacing between the barges, and
4. The mass distribution of the catamaran system

Our analysis on the catamaran system indicates that metacenter of the combined system follows a closed form solution that can be expressed graphically as follows:

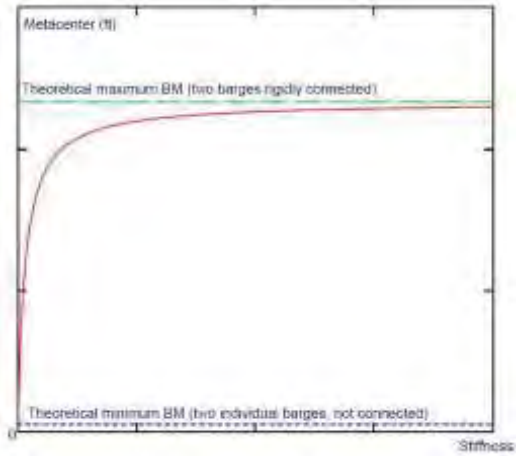

Figure 14 Metacenter of a catamaran system

Fatigue and other Structural Consideration

Due to the short duration of the planned voyage and the low wave loads, fatigue was not considered for this project. If the routing involves blue water tow and long duration, the fatigue issue must be taken into consideration.

Structural response of the barge-bridge system under wind and wave loads must be carefully studied for projects involves blue water tow. Some of the common structural issues may include, but are not limited to: interaction between barge(s) and bridge structure; temporary supports for bridge structures; lashing design, barge stability and strength, cargo (bridge) strength and stability. These issues are more pronounced if the bridge is not completely assembled.

Bridge structure may experience much higher loads during blue water tow than design service loads.

These load are; wind load, wave loads and impact load. The wind and wave induced dynamic loads are cyclic. These loads may cause low cycle fatigues issues that are usually considered in the bridge design by the engineering of Record of the permanent structure. The Construction Engineer retained by the

Contractor shall work with the Engineer of Record to properly address these issues.

Lashing design that follows the concept of base isolation may be able to reduce the dynamic loads on the bridge structure. This method was used successfully by UrbanTech when designing the blue water tow of the 3 Avenue Bridge superstructure from Alabama to Manhattan. Another approach to this problem is to take the potential fatigue issue into consideration during the initial design phase. This approach may not be feasible in US since the construction means and methods are the contractor’s responsibility.

Bridge structures usually are supported at locations other than their permanent supports during transportation. Changing boundary conditions under construction loading will alter the stress patterns.

More often than not, bridge structures will experience higher stresses during transportation than its ideal permanent conditions. The structural analysis must be performed to reflect the structural conditions at each and every construction stage. FE model shall include all structural elements, including falsework and vessel structures to simulate the actual boundary conditions and relative stiffness of the supporting structures.

When bridge structures are transported using barges or other vessels, the geometric nonlinear effects must be considered to include the second order effects, especially when analyzing the stability of the system.

Additional Research

Standard specifications and codes offer very limited coverage on this branch of engineering practice.

Engineering fundamentals shall be applied in designing operations of this type. Research shall be extended to cover the engineering design aspects of ABC practice and to develop standards and code to provide guidance and establish regulations.

Some of the basic research topics, such as cargo structure-vehicle (SPMT) interaction, cargo-vessel interaction and other similar topics may help the engineer to develop more efficient ways to implement ABC technology. Standard specifications or guide specifications will provide owners and regulators the tools for easier implementation of ABC contracts and improve the quality of the final products.

Conclusion

The construction of the main swing span of the Willis Avenue Bridge was delivered using ABC technology. The main truss span and the machinery of the swing span was fabricated, assembled at an off-site facility, and delivered to the job-site in its entirety. This approach allowed the key component of project to be built off the critical path and away from busy Manhattan Traffic. The result is better quality product, shorter construction duration, and less disruption on local economy and people's daily li ves. The analysis method and engineering technology developed for this project greatly advanced the application of ABC technology in the region. AISC gives its 2014 NSBA Accelerated Bridge Construction award to the Willis Avenue Bridge Project.

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G-14: ABC BRIDGE DECK CASE STUDIES

PRECAST GRID DECK PANELS USED TO ACCELERATE REHABILITATION OF I-20 BRIDGES IN GEORGIA

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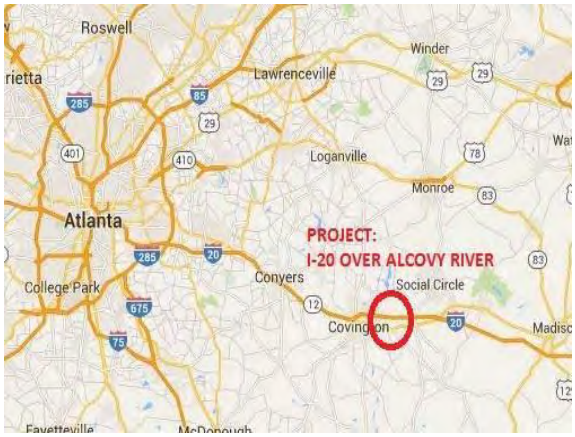
ABSTRACT

Precast grid reinforced concrete & Exodermic® bridge deck systems are not only used for accelerating rehabilitation of large iconic structures. In fact, well over 90% of projects where grid decks are used for weight savings or to speed construction involve structures with less than 50,000 square feet of deck area. At the previous year's ABC Conference a paper was presented on "Overnight Deck Replacement of the I-190 Grand Island Bridge," which involved nearly 100,000 square feet of new precast grid deck panels. The focus of this presentation will be how precast grid panels helped an owner keep traffic moving on a smaller scale deck replacement project.

Georgia DOT I-20 Precast Grid Deck

INTRODUCTION & PROJECT LOCATION

The Georgia Department of Transportation (Georgia DOT) maintains a network of roads and bridges that carry some of the highest traffic volumes in the country, and often employs accelerated bridge construction products when rehabilitating these busy structures. So it's no surprise that when accelerated deck replacement is necessary, the Georgia DOT has used precast grid reinforced concrete decks that allow



various MPT options to help keep traffic flowing during construction on some of the State's heaviest traveled routes like I-75 & I-285 in Atlanta.The rehabilitation of two eastbound and two westbound structures spanning the Alcovy River and Alcovy Watershed on Interstate 20, in Newton County just 35 miles east of Atlanta is the most recent example where a precast grid system was used in Georgia to minimize traffic impacts due to construction. This stretch of I-20 is a major route for more than 40,000 vehicles (roughly 17% truck traffic) per day,connecting Atlanta to Augusta and other popular eastern destinations. A viable detour route was notavailable to permit full bridge closures, so with the interest of the traveling public in mind the DOTrequired staged construction with the bridges fully open to traffic during peak travel times. In late 2013,the Georgia DOT awarded Massana Construction, Inc. of Tyrone, GA a $6.7M rehabilitation contract that included deck replacement, painting and other miscellaneous repairs to the four bridges.

DECK REPLACEMENT

Realizing the highest traffic volumes on this stretch of I-20 occur during weekends and holidays, the Georgia DOT only permitted lane closures from 12:00 noon on Sunday until 10:00am on Friday to eliminate disruptions to weekend travelers. The contract also included a $500/hour penalty for late openings. To reduce weekday traffic congestion during construction, only one lane could be closed at any time on either the eastbound or westbound structures. Staged construction with short five-day closures was possible using a precast grid reinforced concrete bridge deck system. The deck system incorporated two different panel-to-panel connection details depending on the position of the panel within the 192 foot long three span continuous superstructure unit. One detail used a simple shear key splice, and the other used overlapping shear studs in the negative moment regions over the piers to develop tension in the panels. Rapid setting concrete was used for the closure pour areas over the supports and between panels, allowing traffic to be put back on the fully opened bridge less than one day after the pour.

Closure Pour over Supports & Between Panels

A total of 208 galvanized grid deck panels were supplied by BGFMA fabricating member Bailey Bridges, Inc. out of Fort Payne, Alabama. The panels were precast less than 60 miles from the project site at Massana

Construction's precast yard in Fairburn, Georgia using very little formwork which kept costs to a minimum. Including the price of the precast grid deck panels, the total installed deck cost was only $85 per square foot. As expected there was a short learning curve for the contractor setting the precast grid deck panels, but after the first few weeks they were up and running at full speed and able to replace roughly 4,300 square feet (26 panels) in each closure. The project was completed without any major issues and Massana was able to replace nearly 34,000 square feet of bridge deck from April 21 to August 11, 2014. In this 17-week timeframe, four weeks were lost to holidays and inclement weather, and another four weeks were required for MPT work, leaving only nine Sunday to Friday closure periods for deck replacement activities.

CONCLUSION

The Interstate 20 project in Georgia is just one of many small to medium sized projects that take place every year where a precast grid reinforced deck has helped speed construction and/or kept traffic congestion to a minimum by allowing various staging options during deck replacement. The precast grid deck systems are much lighter and more cost effective than conventional precast deck panels while offering several other benefits as well. One of the most significant benefits of precast grid deck systems is the simplicity of panel connection details that do not require post-tensioning - thus saving significant time in design, detailing, fabrication and most importantly installation.

ROUTE 37 MATHIS BRIDGE EASTBOUND OVER THE BARNEGAT BAY – DECK REPLACEMENT AND REHABILITATION

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ABSTRACT

The 4,860-foot long, 60-year old Eastbound Bridge carrying NJ Route 37 is an important evacuation route and major link from the mainland to the popular beach communities in Seaside Heights, NJ. The deck in the 65 approach spans and double leaf bascule span are at the end of their useful life. Deck replacement and rehabilitation work affecting traffic can only be performed in the off-peak season between November and April so that all lanes are available for the summer peak traffic volumes. Approximately 176,000 SF of precast Exodermic deck is proposed for off-season installation, while EB traffic is shifted to the parallel westbound structure. Key project goals include speed and quality of construction through off-site prefabricated components, reduced Life Cycle Cost, and repetitive/efficient details, while maintaining peak seasonal traffic.

INTRODUCTION

The Route 37 eastbound bridge over the Barnegat Bay currently carries three lanes of traffic, 28 feet wide curb to curb. The bridge is posted at an advisory speed of 40 mph, due to the narrow travel lanes. The substandard deck geometry has resulted in the bridge's functionally obsolete classification and the bridge is deemed structurally deficient due to the poor condition of the deck slab which exhibits structural deterioration and the effects of aging in a harsh marine environment.

The proposed improvements to the Mathis Bridge include a total bridge deck replacement, safety improvements to the obsolete traffic gates and railings, substructure and structural steel repairs, bearing replacement, minor painting, and mechanical/electrical upgrades to machinery/controls. The Mathis Bridge will be improved to provide 10-foot wide lanes and a one foot left buffer between the left lane line and the traffic railing, thereby increasing the total curb to curb width by three feet.

Route 37 is one of the main vehicular routes from several townships east of the Barnegat Bay to the mainland. It is also the best emergency evacuation route when both the EB and WB bridges allow unidirectional WB flow. The shore resorts are important summer tourist destinations for the region and thus vital to the local economy.

The project takes advantage of the parallel westbound Tunney Bridge to accommodate the traffic staging.

The Tunney Bridge is not scheduled for any work during the deck replacement project other than off-seasonal temporary traffic shifts during construction staging. The availability of this structure, and the lower off-season traffic volumes allow all Rt 37 traffic to be diverted to the westbound bridge between November 1st and April 30 of each year. A movable construction barrier will separate EB and WB traffic during the off season shifts, allowing full access for work on the Mathis Bridge deck. The colder weather construction season coupled with the access allowed provides an ideal opportunity for precast construction.

Navigation will be maintained throughout the construction, but without openings of the moveable span, between early-December and the end of March for three seasons. Figure 1 shows the two parallel structures, Mathis Bridge to the south, and the Tunney Bridge to the north shown in orange to demonstrate the temporary traffic flow configuration during construction.

Figure 1 - Aerial view of bridges, west half at top, east half bottom

BRIDGE DESCRIPTION

The Route 37 Eastbound Bridge over Barnegat Bay was constructed in 1950. This sixty-six span (sixtythree approach spans with a double leaf bascule main span and adjacent anchor/flanking spans) bridge is 4,877-feet long. The Mathis Bridge originally carried both eastbound and westbound traffic until

1972 when the adjacent high-level fixed bridge (J. Stanley

Tunney Bridge) was built, approximately 12 feet to the north of the eastbound bridge, to carry westbound traffic. Three lanes are currently carried on the westbound roadway and westbound bridge though this corridor. The Tunney Bridge's lane configuration consists of13'-0" outsides lanes and a 12'-0" middle lane with no shoulders (38'-0" curb-to-curb width).

On the Mathis Bridge, the existing outer lanes are 9'-0" with no shoulders and the middle lane is 10'-0" (28'- 0" curb-to-curb width). A 3-foot wide safety walk is located on the south (right) side and a 2-foot wide safety walk is located on the north (left) side. At four locations on the bridge, the roadway is wider by 8 feet on the south side for emergency pull-offs and bridge operator parking areas. In Spans 2E and 2W (the flanking spans), the south sidewalk is wider to accommodate the original rolling barrier gates. The bridge is posted at an advisory speed of 40 mph, due to the narrow travel lanes. The sixty-three approach spans are each 71'-6" long and consist of a simply supported, rolled multi-stringer superstructure that supports a 7 ¼" reinforced concrete cast-in-place, non-composite deck slab (Figure 2).

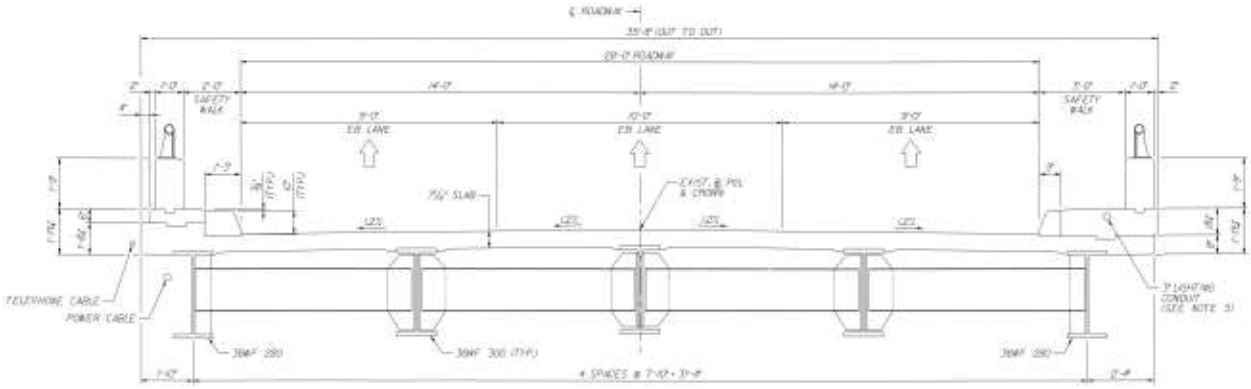

Figure 2 – Typical Approach Span Existing Section

At Spans 8E, 8W, 22W, and 36W only, these four approach spans are wider on the south side for emergency access and bridge operator parking (Figure 3).

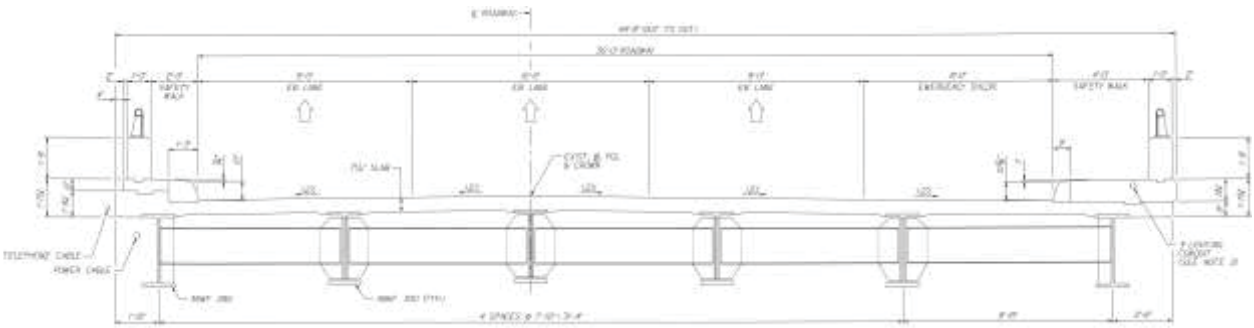

Figure 3 – Typical Parking Span Existing Section

The flanking spans span over the bascule leaves' counterweights and measure 82 feet between centerline of bearings. The flanking span deck is a 6 ¾" thick reinforced concrete non-composite slab. Each flanking span has two longitudinal riveted plate girders that are spaced at 29' -3" on center with a combination of rolled and riveted plate girder floorbeams supporting the rolled stringers under the deck. The bascule span is a riveted plate girder floorbeam-stringer superstructure that supports a steel open grid deck. In the two flanking spans and the bascule span, the two riveted plate girders are internally redundant but structurally and load-path non-redundant and considered fracture critical. The bascule span crosses over the Barnegat Bay, which is a tidal waterway and provides an 80-foot horizontal clearance between existing fenders. The span provides a minimum vertical clearance over the navigation channel of 29.40 feet in the closed position.

EXISTING BRIDGE CONDITIONS

The Mathis Bridge is classified as "Structurally Deficient" due to the poor condition of the deck. As determined during the latest bridge inspections, the structure has a Sufficiency Rating of 55.7 out of 100, with the superstructure and substructure rated in "Fair" condition. The steel stringers were still structurally acceptable with pitting type section losses and non-functional bearings throughout. Other than the bascule and flanking spans, no other steel stringer replacement was warranted, hence the primary bridge work item is the deck and bearing replacement. Recently, the approach spans steel was sand blasted, and painted.

Rehabilitation work includes total deck slab replacement in the main bascule span and all approach spans, substructure and superstructure repairs, bearing replacement, mechanical and electrical upgrades.

PROJECT HISTORY

The original intent of the deck replacement was to stage the construction on the Mathis Bridge in two stages, maintaining one eastbound lane on both the Mathis and Tunney Bridges. However, traffic counts and analysis favored complete off-season closure of the Mathis Bridge, and early approvals by the locals led to this major shift in the project approach to construction staging. At one point the project was briefly considered under President Obama's American Resources Recovery Act (ARRA) with splitting the project into two or three contracts. Subsequently, only the total recoating program was included in ARRA and the major rehabilitation design work proceeded and as deck replacement options were being solidified, a complete bridge replacement option was also required to be considered with a detailed Life Cycle Analysis (LCA) of the various options. Although the LCA indicated the best return for the taxpayer dollar would be a new high level structure, the lack of up-front funding required Final design to continue with the originally scoped total deck replacement.

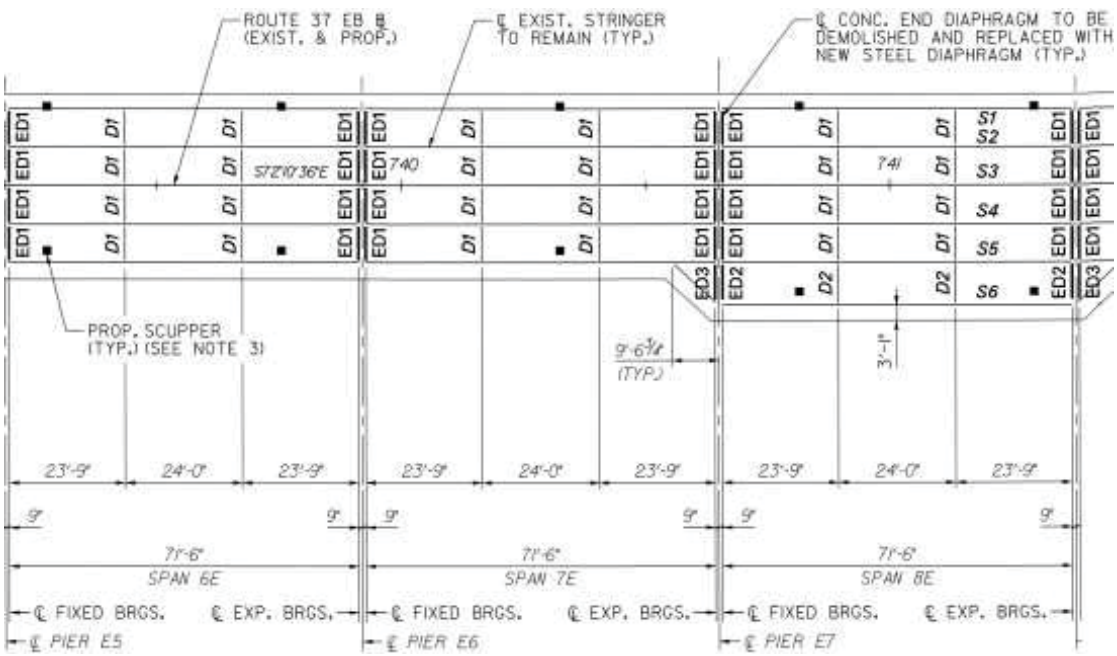

Figure 4 – Existing Framing Plan depicting a standard span, widened parking span, and intermediate/transition span

DECK ALTERNATIVES STUDY

The following deck options were studied: Conventional cast-in-place (CIP) concrete; Precast deck panels with full depth concrete; Partially precast/partially cast-in-place panels; Exodermic deck - CIP and precast;

Half-filled grid deck - CIP and precast

Summary of Deck Alternatives

The selected deck replacement alternative would need to minimize the impact to the bridge profile/elevations, avoid increasing the dead load of the structure, allow a rapid construction sequence, and provide a cost effective solution (note that the costs presented in this section are bare deck costs only for comparison). As the construction was to be performed only in the off-season between November and the end of April each season, the restrictions of cold weather work is a key factor. Since the existing superstructure is non-composite, all proposed options were considered to be made composite to increase the live load capacity of the rehabilitated structure.

Nightly closures of the EB Mathis Bridge for deck replacement in small precast sections was considered briefly but not pursued due to the narrow width of the roadway, profile changes and matching new to existing deck sections and also the need to maintain all three eastbound lanes during the night time in summer.

1. Conventional Cast-In-Place Concrete Deck – Standard NJDOT Slab thickness

Based on NJDOT's standard deck slab design and cover the deck would be 9½" thick and with a positive haunch the total thickness over beam would be up to 11½". The impacts for this deck type are as follows:

- The top of deck elevation would rise approximately 4½".
- Utilizing normal weight concrete, a 37% (260 plf per interior stringer) increase over the existing slab dead load would be imposed. With lightweight concrete, the increase would be 9% (64 plf) and each would reduce the live load capacity.
- The estimated initial cost for the deck alone would be $7,550,000.
- Requires an extended construction time and large volumes of concrete work in the cold weather.

2. Conventional Cast-In-Place Concrete Deck – Non-Standard Slab thickness

Alternatively, a thinner, 8.5" deck was considered. Shallower decks were not considered due to minimum cover requirements. The impacts for this deck type are as follows:

- The top of deck elevation would rise approximately 3½".

- Normal weight concrete would increase deck dead load by 24% (170 plf per interior stringer), reducing the live load capacity. Lightweight concrete would match the existing dead load.

- The estimated initial cost for the deck alone would be $7,200,000.

- Would be more flexible than NJDOT standard decks.

- Will require an extended construction time and large volumes of concrete work in the cold weather.

3. Precast Concrete Deck - full depth precast panels

A conventionally reinforced precast panel section, 9” thick, without prestressing or post tensioning, with an overlay was investigated. The impacts for this deck type are as follows:

- The top of deck elevation would rise approximately 4”.

- Normal weight concrete caused a 30% (210 plf per interior stringer) increase in the slab dead load

  and lightweight concrete, a 24% (170 plf) increase, each reducing the live load capacity.

- The estimated cost for the deck would be $9,150,000.

- Reduced construction duration compared to a conventional deck.

Due to truck transportation limits, the dimensions of individual precast panels were assumed to be limited to 8’ wide x 45’ long. Barging in wider segments would be viable, but the shallow depth of the bay would make it less desirable. The need for longitudinal joints (parallel to traffic) could be avoided with only transverse joints (perpendicular to traffic) every 8 feet and an overlay system. Overlay options included a standard 1½” bituminous concrete, a ¼” to ½” epoxy based overlay, a polymeric concrete – methyl methacrylate (MMA) about 3/8” thick, and ¾” polyester polymer concrete.

If the panels were prestressed, a reduced deck thickness of 8” was viable and the impacts would be:

- The top of deck elevation would rise approximately 3”.

- With normal weight concrete, a 17% (125 plf per interior stringer) increase over the existing slab

dead load and with lightweight concrete, a 13% (100 plf) increase with reduced live load capacity.

- The estimated initial cost for the deck would be $9,650,000.

- Reduced construction duration compared to a conventional deck.

A 4. Precast Concrete Deck - partial depth precast panels/partial depth cast-in-place

total deck thickness of 9½" with a 3 ½ -inch precast panel, topped with a 6 inch- cast-in-place portion was considered. While the system offers some time savings, increased dead loading, increased profile and the amount of cast-in-place concrete placement in the colder weather negated this option. The impacts are:

- The top of deck elevation would rise approximately 4½".

- With normal weight concrete, a 37% (260 plf per interior stringer) increase in the slab dead load and with lightweight concrete a 12% (82 plf) increase, either would reduce the live load capacity.

- The estimated cost for the deck would be $8,400,000.

- Will require an extended construction time and large volumes of concrete work in the cold weather.

5. Alternate deck systems - Precast Exodermic Deck

A 7½" deep section precast exodermic deck was investigated for replacement of the existing deck. The weight of the exodermic deck is approximately 85 psf including the CIP fill concrete over the stringers. The impacts are as follows:

- The top of deck elevation would rise approximately 2½".

- If normal weight concrete is used, there would be a slight reduction of dead load and with lightweight concrete the load reduction would be improved. When made composite, either of these options would increase the live load capacity to the desired HS-25 for the approach spans.

- The estimated cost for the deck would be $9,650,000.
- Reduced construction duration compared to a conventional deck.

An overlay system to improve the riding surface added approximately 3/8” to ¾” to the deck depth.

6. Cast-In-Place Exodermic Deck

A 7½” deep cast-in-place precast exodermic deck was also investigated for replacement of the existing deck. The concrete would be cast-in-place and an overlay would not be required and same impacts as above, with the following modifications: follows:

- The estimated cost for the deck would be $8,250,000.
- Reduced construction duration compared to a conventional deck as the steel grid acts as formwork, but a longer duration compared to precast alternatives

7. Partially Filled Steel Grid Deck and Other options

Partially filled steel grid decks were also studied, the concrete portion of this deck can either be precast or cast-in-place similar to the exodermic decks. In discussions with the industry professionals the approximately 8 foot span across the typical existing girder was a concern for fatigue stresses based on the requirements of the current AASHTO LRFD design specifications. Although there may be changes to the AASHTO in the future, at the time of design we chose not to pursue filled steel grid decks options.

Other refabricated superstructure units, considered include the Inverset™ Bridge System, Effideck system and a standard total superstructure replacement. Given the redundant multi-girder framing in all of the approach stringer spans and the fair condition of its structural steel framing and that the recent superstructure maintenance which recoated the steel with a full three-coat painting, the Inverset™ system did not seem justified. An initial cost estimate of the Inverset™ Bridge System was approximately $27 million. The initial cost of replacing the entire superstructure with new, painted structural steel and a conventional concrete deck was estimated to be approximately $23 million. Alternatively, by making the existing non-composite steel stringers composite with a lighter deck system, the approach spans were upgraded to HS-25 live loading.

Deck Type Recommendation

The precast Exodermic deck alternative was agreed upon by the team for the Mathis Bridge Project.

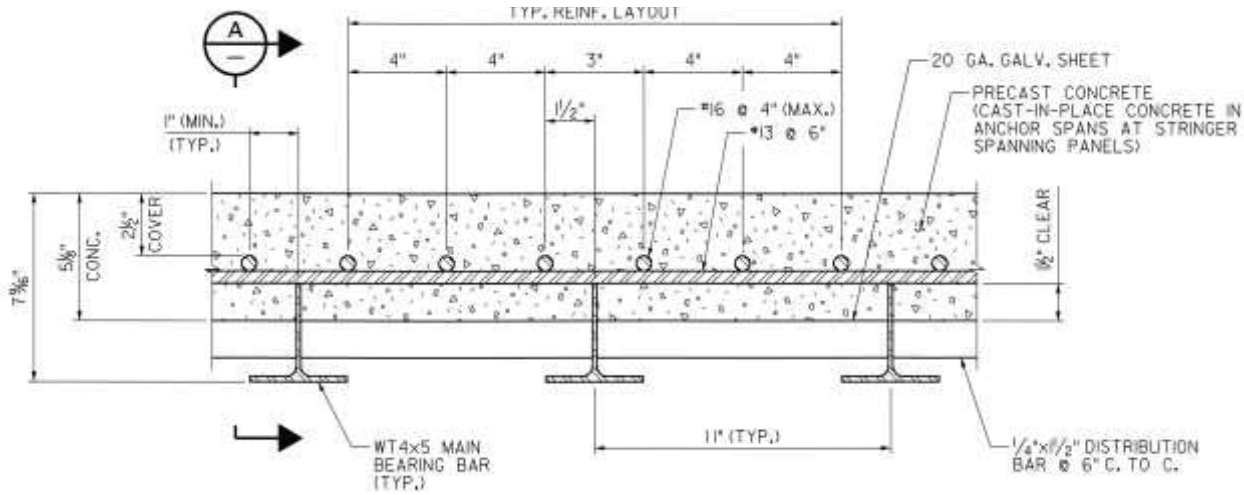

Figure 5 – Transverse precast Exodermic deck section

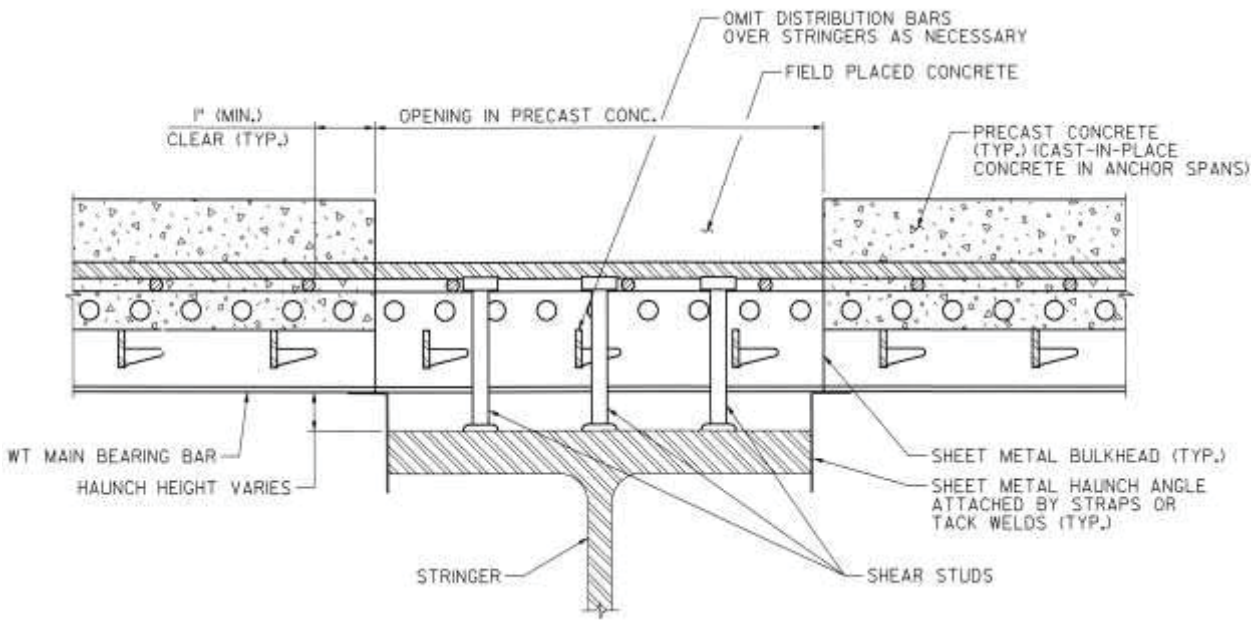

Figure 6 – Longitudinal precast Exodermic deck section including shear studs for composite action

Overlay Recommendation

A ¾" Polyester Polymer Concrete (PPC) overlay was agreed upon by the team. PPC cures very rapidly in varying temperatures and the surface preparation requirements are favorable. PPC also develops a strong bond to the concrete substrate through the use of a reactive penetrating primer.

Closure Pour Material

Several options, including Ultra High Performance Concrete, were studied for use in the cast-in-place closure pours. The closure pours referred to include 8” wide splice joints between panels, 18” wide full depth pours over steel beams to develop composite action, and pours for 18” wide deck joint blockouts.

Ultimately, a general performance specification was used for the closure pours to allow the contractor to use the material best suited for their means and method that met the design requirements and maintained the allotted construction schedule. The contractor was also allowed to use different materials throughout the project. For example, a quick set concrete could be used when temperatures are favorable and an MMA High Performance Grout could be utilized during the coldest weather since it can be placed in extreme temperatures. Eliminating the need for temperature control measures could offset the higher base unit cost of the material in this case. Performance requirements for the closure pour included the final strength, intermediate strength needed prior to loading, material specific allowable temperature ranges for placement, and shrinkage limits. This was done to ensure a cost effective solution that will benefit the contractor and the NJDOT.

REPETITIVE DETAILS FOR REPETITIVE SUCCESS

Maintaining the most repetitive details that can be reproduced systematically by the fabricators and the Contractor was a major goal for this project. The bridge is on a tangent horizontal alignment and the majority of the bridge section is identical in span length, stringer spacing and cross sectional characteristics with the exception of the bascule span and two flanking spans. There are also four spans along the bridge which are wider to accommodate emergency pull-off areas and bridge operator parking. The spans immediately adjacent to the parking spans have minor differences as a result of the transition. The details below demonstrate the repetition carried out between the approach spans and the parking spans. All scuppers, railings, junction boxes, and lighting standards were located in the same spot within an individual precast panel and within a given span wherever feasible.

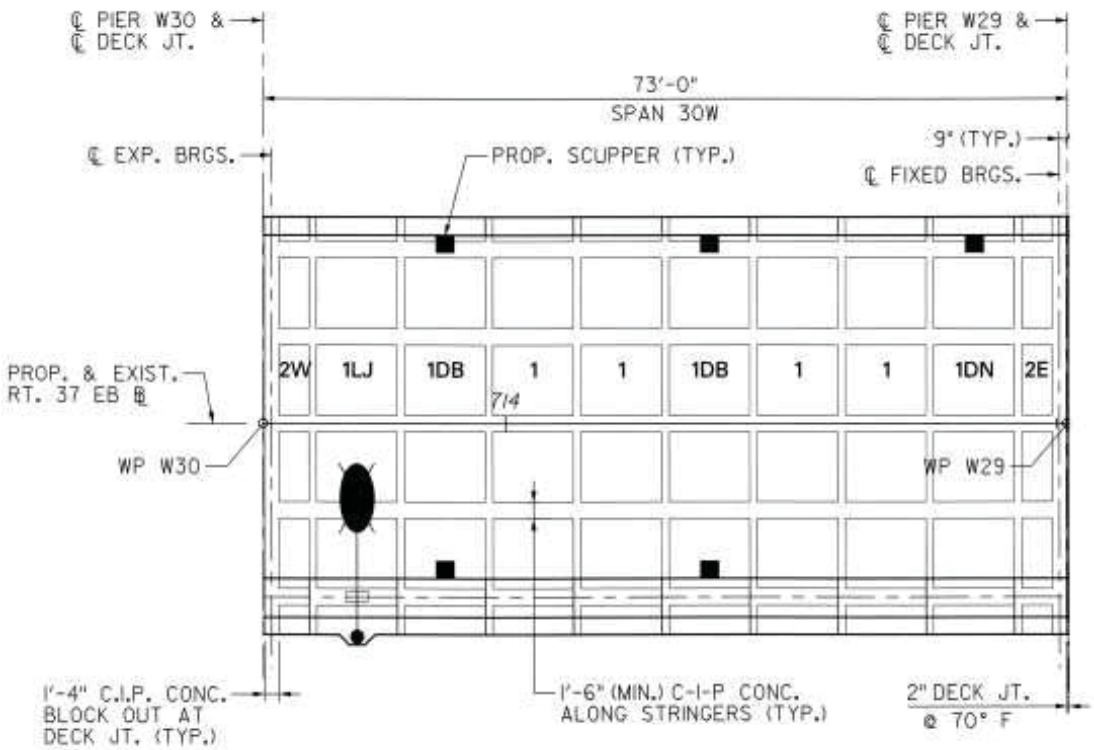

Figure 7 – Layout for a typical span

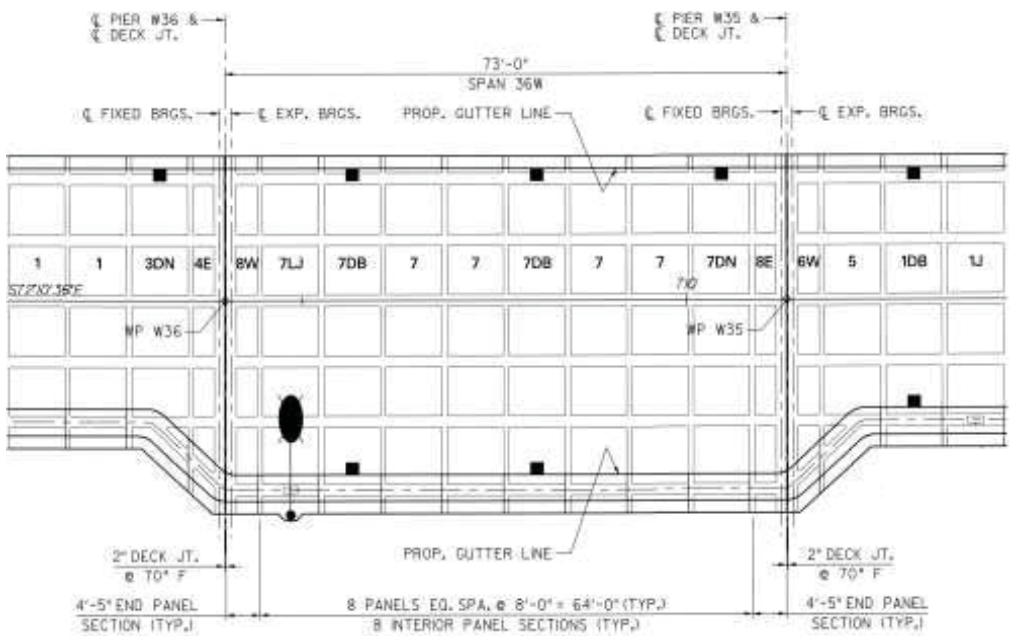

Figure 8 – Layout for a parking span and the adjacent transition zones

The standard type '1' panels shown include minor modifications for embedded elements such as drainage scuppers (DB, DN, or DS) and lighting standards (LJ) and the type two panels make up the closure portions near the deck joints. Between these two basic precast deck panel types, over 150,000 sf of the

precast panels are accounted for, or about 85% of the precast deck area proposed for the project. Prefabricating the steel railing system and baseplate with anchor bolts set in the precast panels is expected to significantly shorten the construction time for bridge railing work. This railing system, crash rated for TL-4 loading, also closely replicates the existing open steel railing system, while the standard post and steel tubular rails lend themselves to a fast, and high quality off-site fabrication.

CONSTRUCTION STAGING

To arrive at the optimum staging plan, traffic studies during peak season and off-peak season were undertaken along with evacuation studies. In addition to the technical studies, the findings were relayed to public officials through several meetings which included local officials from Seaside Heights Borough, Seaside Park, Lavallette, and Toms River. One of the original concepts for staging included maintaining two lanes of traffic in each direction on the Tunney Bridge, requiring substantial modifications to it. However,

a revised staging plan was devised based on traffic counts and studies that supported two lanes westbound and one eastbound in the off-season. Evacuation analysis with several scenarios, also supported this revision. In addition, a considerable amount of time and preparatory work on the Tunney Bridge could be avoided. Figure 9 shows the temporary traffic configuration on Tunney. This revised staging plan lead to an overall reduction in the estimated construction schedule by one to two years. Access to the existing businesses would also remain open under this plan.

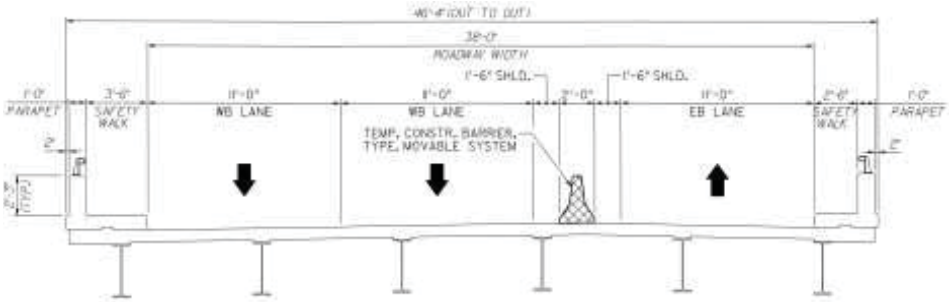

Figure 9 – Temporary traffic configuration on Tunney Bridge during Mathis Bridge work/closure

The summer traffic patterns of three lanes in each direction on the bridges would also be maintained. This ensured the construction would not impact the peak summer travel which is vital to the local economy. Strict provisions were included in the documents which require the Contractor to provide smooth transitions where the deck changes from the existing section to the new section (Figure 10), and to make sure the bridge was opened up to all three eastbound lanes prior to the peak season.

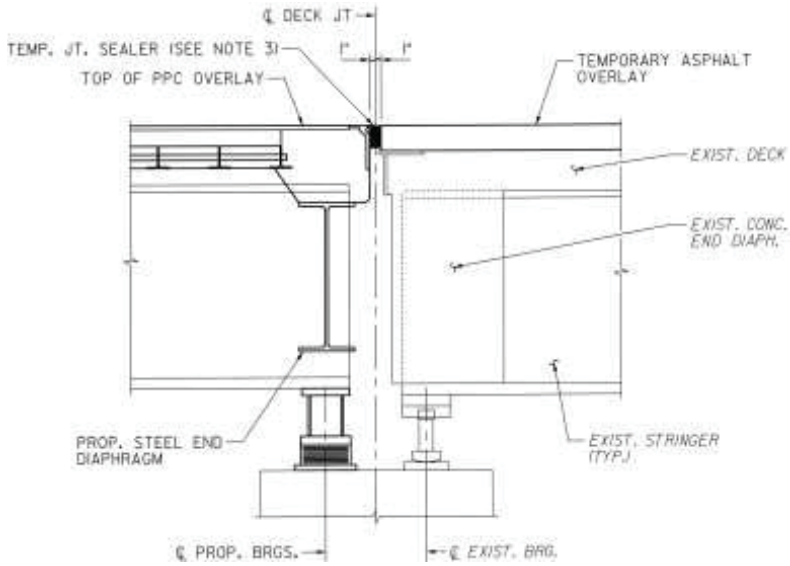

Figure 10 – Transition detail from new deck section to existing deck section

DECK ITEM BID PRICES

The winning Contractor's bid prices relevant to the bridge deck only are shown in Figure 11 below, at about $14 million or about $79/SF including shear studs. Note that the bare Precast Exodermic deck costs at bid were very similar to the estimate from the deck study. It is interesting to note that the precast Exodermic deck unit costs are less than the cast-in-place costs, apparently due to the repetitious design details and a large quantity of repetitious fabrication. Even if the cast-in-place closure pours are accounted for, which adds $7.50/SF to the precast panels, the precast panels are less costly even though their bid price includes all reinforcement steel whereas the reinforcement steel is paid for separately for the cast-in-place items.

DECK ITEM DESCRIPTION	UNIT	CONTRACT QUANTITY	UNIT COST	TOTAL COST
SHEAR CONNECTOR	UNIT	83,000	$7.22	$599,260.00
CAST-IN-PLACE EXODERMIC BRIDGE DECK SYSTEM, HPC	SF	6,179	$67.71	$418,380.09
CONCRETE CLOSURE POUR	CY	1,774	$730.21	$1,295,392.54
PRECAST EXODERMIC BRIDGE DECK SYSTEM	SF	176,076	$54.22	$9,546,840.72
POLYESTER POLYMER CONCRETE OVERLAY	CF	9,082	$224.59	$2,039,726.38

Figure 11 – Deck related costs

CLOSING

As of the date of this paper, the first off-season bridge closure is scheduled to start November 1, 2015. All steel grid shop drawings have been approved with very minor changes to the design documents and the

precast panel shop plans are under review. No major change to the design documents has been submitted to date but several small fabrication changes have been accepted. The

Contractor's baseline schedule plans to complete all the bridge deck work in just two seasons as opposed to the three allowed in the project documents. The design team is aggressively reviewing shop plans and submittals via an electronic media. The next ABC conference may allow us to share additional construction challenges and how they were overcome. It is anticipated that the design approach and details will help achieve the shorter schedule proposed by the Contractor.

REPLACEMENT OF FULL-WIDTH CROSS GIRDERS ON PULASKI SKYWAY DURING WEEKEND CLOSURES EXTENDED ABSTRACT

BY: Scott Thorn, New Jersey Department of Transportation (Project Manager)
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The Pulaski Skyway (i.e., the Skyway) is a 3.5 mile long viaduct that connects the cities of Jersey City and Newark in New Jersey that was completed in November 1932, and is owned by the New Jersey Department of Transportation (NJDOT). It is listed on the National Register of Historic Places. The

Skyway crosses over the Hackensack River, the Passaic River, the Hackensack Meadowlands, and various railroads and roadways, including the New Jersey Turnpike. Before the recent partial closure for deck replacement, the Skyway was carrying approximately 70,000 cars / day. The northbound roadway of the Skyway leads to the Holland Tunnel, a major crossing into lower Manhattan.

The majority of the bridge structure consists of pairs of cantilevered deck or subdivided-Pratt through trusses with curved lower chords. The main river crossings are 1250 feet long cantilever through trusses and suspended spans. The westernmost 500’ of the viaduct and the easternmost 3500’ length of the viaduct originally consisted of concrete encased deck girders and stringer spans supported by encased steel column bents. The steel column bents consist of steel cross girders (i.e., steel cap beams) bolted or riveted to the top of steel columns. The Easternmost low level section originally supported an approximately 16” thick two course concrete deck.

South Elevation of Skyway, looking toward northwest toward Hackensack River Spans

Background of Deck Replacement Project:

Starting in 2007 a Concept Development Study and Feasibility Assessment was performed and concluded that rehabilitating the Skyway is the preferred alternative. The rehabilitation work will be done under several contracts. The $400 million deck replacement currently underway presented many challenges during design, the biggest of which was the construction schedule. One million square feet of deck will be replaced within a 30 month period. Because there is a major alternate route to the Holland Tunnel, and because it is paramount to ensure that traffic can exit from lower Manhattan, "southbound" or outbound traffic from Manhattan will be kept open, while "northbound" or inbound traffic will be closed for the duration of deck replacement.

In order to meet the construction schedule, the contractor will need to replace more than 10,000 square feet of deck per week. It was determined that using the accelerated bridge construction practice of installing roadway panels fabricated and cast offsite, and covered with a nominal 1" thick Polyester Polymer Concrete overlay, would be the optimum choice for meeting the schedule. Some of the panels are precast concrete reinforced with stainless steel; the remainder of the panels are precast galvanized Exodermic panels built with galvanized reinforcement. All panels between expansion joints will act monolithically through the use of Ultra-High Performance Concrete closure pours. Due to the importance of this roadway and the difficulties caused by lane closures for future rehabilitation, all new design and construction work on the Skyway was designed for a 75-year service life. Besides the use of offsite fabricated and precast roadway panels, other accelerated bridge construction practices were utilized.

In late October 2012, during encasement removal on existing structural steel, workers discovered large holes in the webs of the east and west cross girders at the double column expansion Bent #3. Cross girders support all of the longitudinal girders at this location and are fracture-critical (FCM) members. All of the original cross girders on the Skyway have a similar design: They are "built-up" members having web plates riveted to pairs of top fla nge angles and to bottom flange angles. A structural analysis was performed on the "As-inspected" condition of the west cross girder, and an HS-3 rating was determined.

Initially, temporary Mabey brand towers were placed below the longitudinal members that framed into Bent #3. At this point, NJDOT made the decisionto replace all of the original cross girders that were located at expansion bents with new cross girders, consisting of a web plate bolted to pairs of top flange and bottom flange angles.

For the cross girder replacement, 10 full weekend closures are permitted. All traffic will be detoured from 9PM Friday night until 5AM on Monday. The first pair of east and west cross girders to be removed and replaced were located at Bent #A2, which occurred before the inbound roadway was closed to traffic. The following sequence of construction tasks had to be performed during this closure:

- (First Weekend): Maintenance of traffic necessary to close Pulaski Skyway and portions of Tonnele Circle;
- Jacking and bearing of the longitudinal girders;
- Removal of existing split median;
- Demolition of existing roadway joint, concrete deck header, concrete safety walks, and concrete balustrade, above existing cross girders;
- Removal of existing rivets connecting cross girders to longitudinal girders, including fascia girders (Note: Some of the rivets were replaced with High Strength bolts in advance of the weekend closure);
- Removal of existing rivets connecting cross girders to column brackets and column cap plates;

- Removal of existing cross girders;

Removal of existing cross girder at Bent A2

- Erection of new cross girders;
- Installation of reinforcing steel for Polyester Polymer Concrete header;
- Partial bolting of new cross girders;
- Erection and clamping of temporary support beams of temporary split median barriers;
- Installation of temporary roadway plates and temporary safety walk plates;
- Installation of temporary split median barriers;
- Re-opening of Skyway to traffic in both directions;
- During the week and at night time, between weekend closures, installation of additional bolts;
- (Second Weekend): Maintenance of traffic necessary to re-close Pulaski Skyway and portions of Tonnele Circle;

New cross girder at Bent A2

- Final bolting required for opening of roadway;
- Removal of temporary split median barrier;
- Removal of temporary roadway plates and temporary safety walk plates;
- Placement of the PPC roadway header;
- Final Installation of temporary split median barrier;
- Placement of Compression Seal Joint in Polyester Polymer Concrete roadway header;
- Re-opening of Skyway to traffic in both directions

Erection of new cross girder at Pier 6

Lessons were learned from the design and construction of the cross girders of Bent #A2 that were applied to the design and construction of the remaining expansion bent cross girders slated for rep lacement. Foremost of these lessons was incorporating design details that enhanced the constructability of all new replacement cross girders. This paper explores in detail the Cross Girder/Floorbeam replacements and other challenges faced by the Pulaski Skyway rehabilitation design and the multiple techniques used to respond to the challenges in order to complete one of the largest accelerated bridge construction projects in the United States to date.

G-15: ABC APPLICATION FOR RETROFIT

LONG KEY BRIDGE V-PIERS REHABILITATION

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INTRODUCTION

The Long Key Bridge is a first generation precast segmental concrete box girder erected using the span by span method of construction. The V-Piers at the expansion joint locations exhibit significant deterioration requiring replacement. This paper discusses the design and construction of the expansion joint V-Piers replacement of this unique structure. The challenges of this project include accelerating construction during V-Pier replacement, maintaining uninterrupted traffic through the bridge and working within the Florida Keys National Marine Sanctuary. The V-Pier replacement is currently the largest rehabilitation project in South Florida.

PROJECT OVERVIEW

The Long Key Bridge is a 12,152 ft. long precast segmental concrete box girder, designed and constructed between the late 1970's and early 1980's. The Long Key Bridge has 103 spans with eight spans per unit. The bridge is divided into thirteen continuous units using external post-tensioning tendons longitudinally. The typical unit has the interior piers supported on bearing pads whereas at the ends of the bridge units, the V-Piers have Freyssinet-type hinges located at the top and bottom of the pier legs.

In order to enhance the corrosion protection of the reinforcing steel in the concrete members, all rebars were epoxy coated.

The bridge foundation uses two 42"-diameter concrete drilled shafts terminating ion 48" deep footers. These footers are monolithically connects to a 48"x48" precast struts. Longitudinal movements of the bridge due to long term deformations, temperature and horizontal loads are accommodated by shear deformation of bearing pads at the base of the interior V-Pier. At expansion joint locations, the V-Pier legs move in opposite directions; this longitudinal movement is absorbed by large rotations of the concrete hinges. Since its early days the bridge expansion joint V-Pier hinges have suffered ongoing corrosion damage,

Figure 1. Existing EJ V-Pier

cracking, spalls and delaminations; severe deterioration is currently present to the point requiring replacement. Figure 1 shows an existing expansion joint V- pier.

The replacement project encompasses the twelve V-Piers at the expansion joints with the following requirements:

- Replacement keeping the same original appearance
- Maintain uninterrupted traffic on the bridge at all times; only evacuation route of the Lower Keys during Hurricane season
- Meet stringent environmental requirements of the Florida Keys National Marine Sanctuary
- Replace bearings at begin and end bridge
- Construction activities not to interfere with Historic Flagler Bridge
- A structural system that accelerates construction resulting in minimum inconvenience to the public.

DESIGN CONSIDERATIONS

Several design considerations were incorporated into the engineering of the replacement project. Since the bridge already had thirty years of service, all long term deformation due to creep, shrinkage and steel relaxation have already been dissipated, so the proposed system only needed to accommodate temperature movements and lateral loads. This was accomplished by a using a precast inverted delta frame that closely matches the existing bridge aesthetics. Elastomeric bearing pads located on top of the delta-frame will support the segmental box girder and manage lateral forces and longitudinal displacements. The proposed precast V-Pier has a pier-cap which is designed with a special moment connection to the existing drilled shafts, providing continuity to the foundation similar to the original design.

The replacement required slight modifications to the statical system without affecting the remaining members of a bridge unit. Since the V-Pier legs connection to the superstructure is no longer monolithic, the bridge is transversely restrained by a shear key that is introduced between the delta-frames. Also, the horizontal force produced by the vertical force transmitted to the V-Pier legs created a dead load plus prestress bending moment decrease of 8% in areas of maximum positive bending. These changes are not detrimental to the performance and load carrying capacity of the bridge.

The temporary support system (TSS), envisioned and engineered to support the box girder during the V-Piers replacement, consists of an innovative steel structure, which is unique, safe and efficient. It has been designed with the characteristics of a permanent structure as it carries the existing traffic while the substructure is being replaced. The TSS is supported on temporary drilled shafts and includes a sophisticated jacking and monitoring system to transfer all dead and live load from the superstructure without perceptible displacements. As the replacement work is done within an ecologically sensitive environment and within the FKNMS, the TSS avoids special permitting efforts by being independent of the existing bridge (not using the existing bridge for support). From the constructability aspects, the TSS is reusable, accommodates existing foundation and superstructure as-built conditions and allows for all assembly and disassembly operations to be done above water.

REFERENCES

1. *SR-5/US-1/Long Key Bridge V-Piers Replacement at Expansion Joints and Deviation Blocks Repairs,Bridge Development Report, FDOT FM No. 25055423201, URS Corporation Southern, August, 2011.*

HYBRID BRIDGE WIDENING PULLS IT ALL TOGETHER

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ABSTRACT

This presentation discusses the design and construction of the widening of an existing cast-in-place (CIP) post-tensioned concrete box girder superstructure using a combination of precast/pre-tensioned and CIP concrete elements that are post-tensioned for continuity. The existing SR303L Bridge over US60 and the BNSF Railway northwest of Phoenix is a four-span, CIP post-tensioned concrete box girder superstructure integral with the piers. In lieu of an "in-kind" CIP widening, the Construction Manager at Risk (CMAR) contractor requested the widening utilize Precast Bridge Element Systems (PBES), specifically precast girders, to eliminate CIP falsework over US60 traffic and reduce construction time. The selected method combined AASHTO girders post-tensioned with CIP concrete pier tables to mimic the structural behavior of the existing bridge. This "Accelerated Bridge Construction" (ABC) approach results in an efficient, hybrid superstructure that meets project goals of minimizing traffic disruptions and reducing construction costs and time.

INTRODUCTION

The US60 (Grand Avenue) SR303L Underpass Bridge illustrates a cost-effective, hybrid concrete solution, substituting precast concrete elements for cast-in-place elements to address some common challenges encountered during urban freeway construction today.

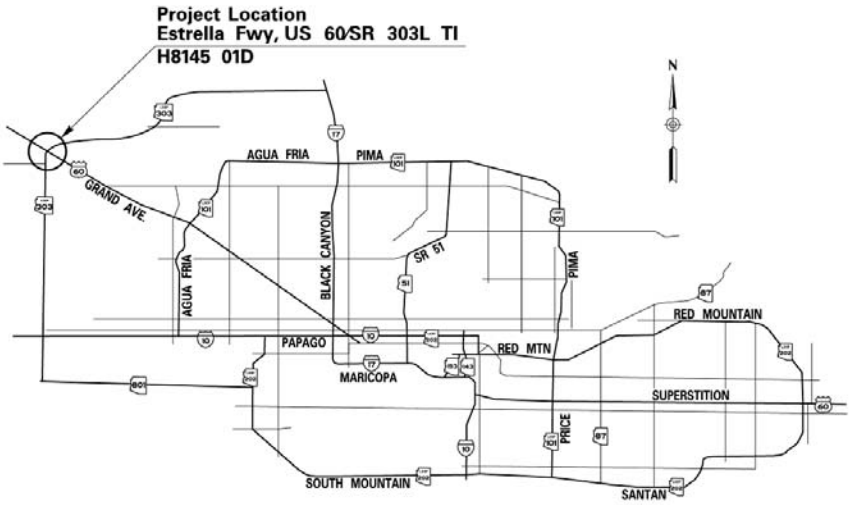

Figure 1 – Phoenix Regional Freeway System and Project Location

SR303L is a 40-mile long new freeway in the western and northwestern portions of the Greater Phoenix Metropolitan Area. It is part of the Regional Transportation Plan Freeway Program (RTPFP) adopted by the Maricopa Association of Governments (MAG) in November 2003 and is a major element of the Regional System in the northwest portion of Maricopa County. It opened for traffic as a two-lane facility in 1992 with the primary purpose of providing an alternate route for US60. Future growth and traffic projections are driving the need to expand the regional freeway facility in the northwest valley.

The challenge of maintaining traffic on US60/Grand Avenue during construction was complicated by the limited detour options. The high traffic volumes of 24,000 and 12,700 vehicles per day on US60 and SR303L respectively (2013 data), precluded extended closures with off-site detours. On-site detours (i.e. at-grade crossings of SR303L) were not practical due to the high speed differential and considerable grade differences. Consequently, the final design of this bridge was driven primarily by the need to maintain both vehicular and railway traffic through this interchange.

The US60/SR303L Traffic Interchange (TI) is a two-level, interim interchange configured as a partial cloverleaf in the southwest and southeast quadrants of the TI arranged to minimize impacts to adjacent existing residential development in those quadrants.

The cross-section for US60/Grand Avenue was designed for an ultimate configuration of four through lanes and two left-turn lanes. SR303L was designed for an ultimate configuration of six northbound (NB) and six southbound (SB) lanes. The final design consisted of a 536-foot long, 197-foot wide, four-span bridge, with a span arrangement of 105’+194’+118’+114’ matching the existing bridge.

Figure 2 - Rendering of the US60/SR303L Interim TI

CONSTRUCTION MANAGER AT RISK (CMAR)

For the construction of the US60/SR303L Interim TI, ADOT decided to use the Construction Manager at Risk (CMAR) alternative delivery method. The CMAR process allows selection of a contractor based on qualifications during the project development process. Once selected, the CMAR becomes a collaborative member of the project team. As the contract documents are developed, the CMAR and the owner (ADOT) negotiate a Guaranteed Maximum Price (GMP) the owner agrees to pay the contractor for the completed project. The CMAR's involvement in the design of the project provides a means to allocate the risk between the construction manager and the owner, eliminating or greatly reducing change orders during construction.

A HYBRID OPTION

The existing US60/SR303L Bridge was constructed in 2000 for the Maricopa Department of Transportation. Ownership was subsequently transferred to ADOT. The bridge is a four-span, CIP posttensioned concrete box girder superstructure, with integral piers and a precast drop-in span over the railroad. It carries SR303L over US60 (Grand Avenue) and the Burlington Northern Santa Fe (BNSF) Railway.

Figure 3 – Existing US60/SR303L Bridge

This project required widening the existing bridge approximately 120 feet, from an out-to-out width of 73’- 8 to 197’-1, about 2-½ times its original width. The original DCR concept called for symmetric widening on both sides of the existing. Subsequent roadway realignment by the design engineers allowed for the much simpler south side only widening shown below. Pier locations, span arrangements and joint locations were all dictated by the configuration of the existing bridge.

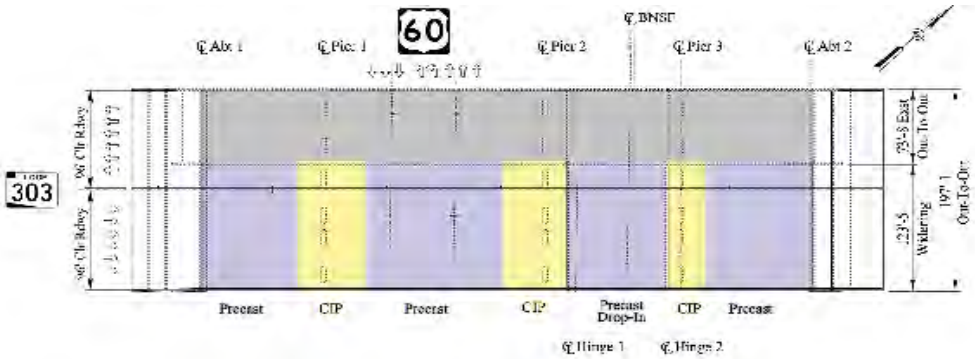

Figure 4 – Plan View of Bridge

A conventional approach would have been to widen with an identical CIP post-tensioned concrete superstructure. The 23 feet plus of vertical clearance over US60 due to the proximity of the BNSF tracks created ample vertical clearance for falsework over traffic. At 194 feet, the length of Span 2 was beyond the typical capabilities of conventional precast AASHTO girders used and available in the Phoenix area.

CIP concrete however comes with a number of key issues:

- Falsework construction is costly and time consuming.
- CIP concrete forming and reinforcing placement are time consuming and critical path items.
- Falsework carries additional risk that is magnified when over live traffic.

- Falsework opening limitations constrain existing roadways.

- Traffic closures are required for large volume concrete pours over US60.

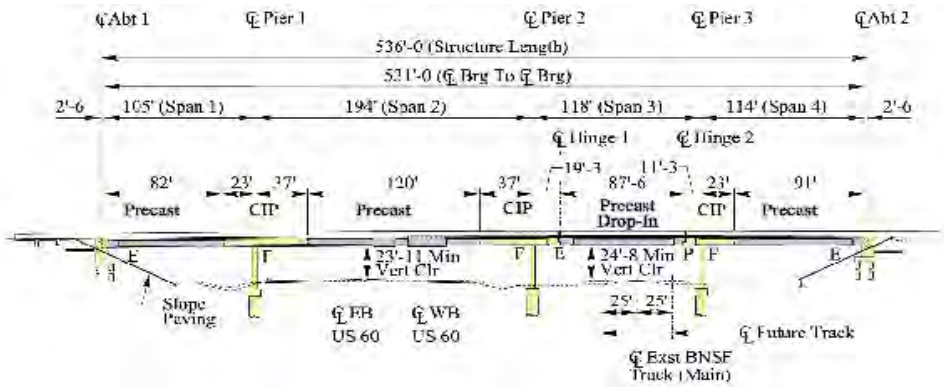

Figure 5 – Elevation View of Bridge

Working with the CMAR contractor, an improved method for the bridge widening combined precast AASHTO girders, post-tensioned with CIP concrete pier tables. CIP concrete was used near and over the piers where structure behavior compatibility was more critical. 36,000 square feet of CIP concrete bridge superstructure was replaced with precast elements (PBES) and stay-in-place deck forms in mid-span areas to minimize disruption and risk to existing traffic. This method eliminated large segments of falsework, most importantly falsework over US60. The two structure types (CIP and PBES) were spliced and post-tensioned to provide continuity and comparable structural behavior to the parent bridge. Spliced connections between CIP and precast elements were located where dead load moments were low and where shoring towers could be placed without significant impacts to US60.

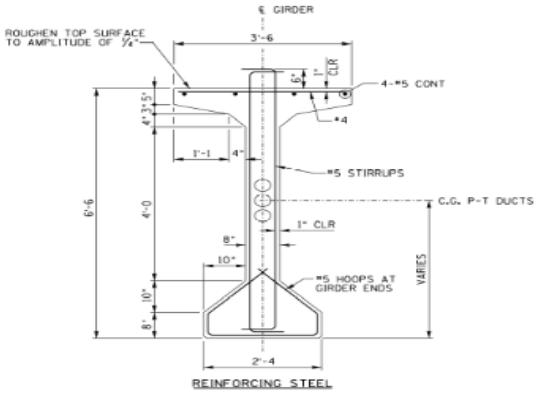

Figure 6 – AASHTO Type "Super" VI

The structure depth of the existing bridge is 7'-4". AASHTO Type "Super" VI girders were selected for the widening, resulting in a 7'-5" section depth. The slightly deeper section did not compromise minimum vertical clearances. (The Type "Super" VI girder is identical to a standard Type VI except for the height. The "Super" VI girder has an additional 6" of web height for a total girder depth of 6'-6".)

The AASHTO LRFD Bridge Design Specifications, 6th Edition, Section 5.4.6.2 limits the size of the PT duct to 40% of the gross concrete thickness at the duct. Based on our research previous ADOT projects successfully used spliced girders with 8” webs and 4” nominal PT ducts. It was also noted that the PCI

Bridge Design Manual, 3rd Edition, Section 11.4.5.1 listed multiple state agencies, existing bridges and numerical examples in which this AASHTO criteria was not met.

The shear capacity of the girder was checked and found to be adequate for the loads prior to grouting of the PT tendons. Therefore it was determined that the 8” webs would function adequately and be more cost effective than increasing the web width.

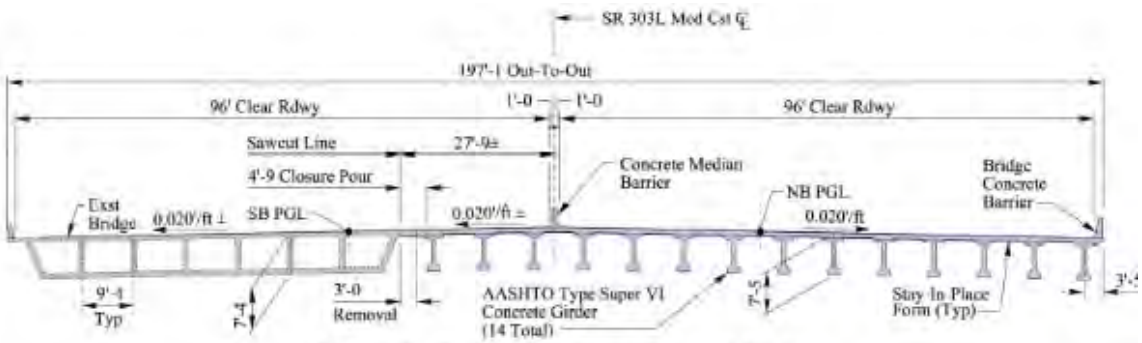

Figure 7 - Typical Section at Precast Girders

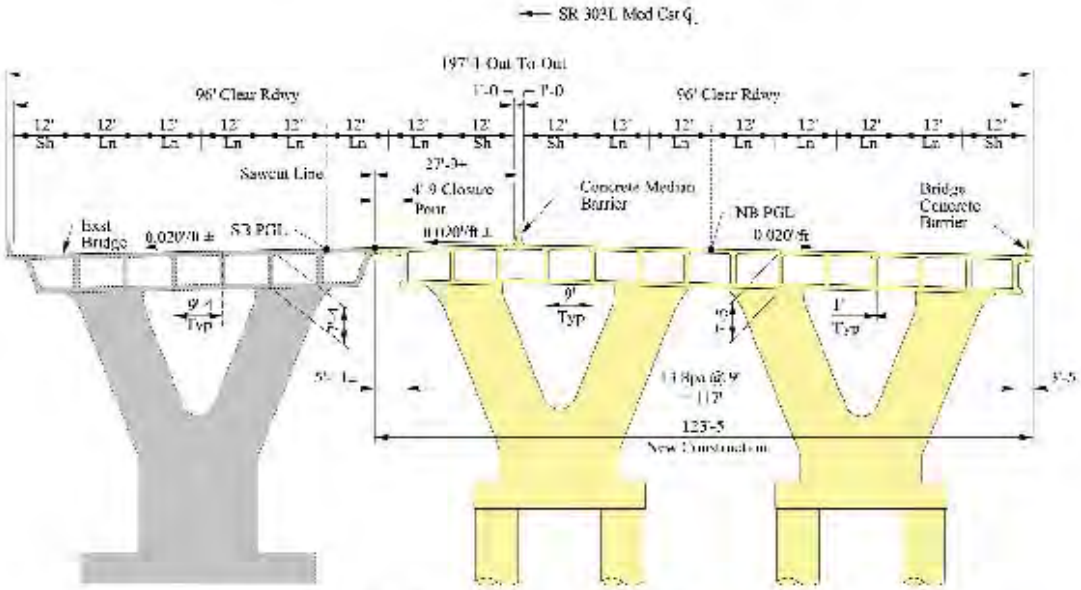

Figure 8 – Typical Section at CIP Box Girder Pier Tables

Joining the new and existing structure was accomplished with a 4’-9± wide by 1’-4± thick deck closure pour after completion of post-tensioning. The existing deck overhang concrete was removed saving and

exposing the existing deck reinforcing. New deck reinforcing was spliced with the exposed existing reinforcing and additional reinforcing was added to account for the additional thickness and strengthen the connection between the existing and the widening.

Figure 9 – Existing Reinforcing at Deck Closure Pour

The initial pre-stressing utilized 0.6” diameter straight strands to carry the self-weight of the girder, the plastic deck concrete and a nominal construction load. Harped strands could not be used due to the draped PT ducts installed in the girder webs. It was also desirable to minimize initial girder camber to allow for the additional camber from the second stage post-tensioning and to control screed grades. After the deck pour, the minimal initial pre-stressing produced about 0.5” upward camber in the girder prior to post-tensioning.

Figure 10 – PT Ducts in Precast Girder Web

The 6'-6" deep girders combined with the relatively short interim spans (the interim span length from shoring tower to shoring tower prior to post-tensioning), required minimal pre-stressing to accommodate the initial loads. All precast girders were designed for two-stage stressing, an initial pre-stress for dead loads, and post-tensioning for final loads.

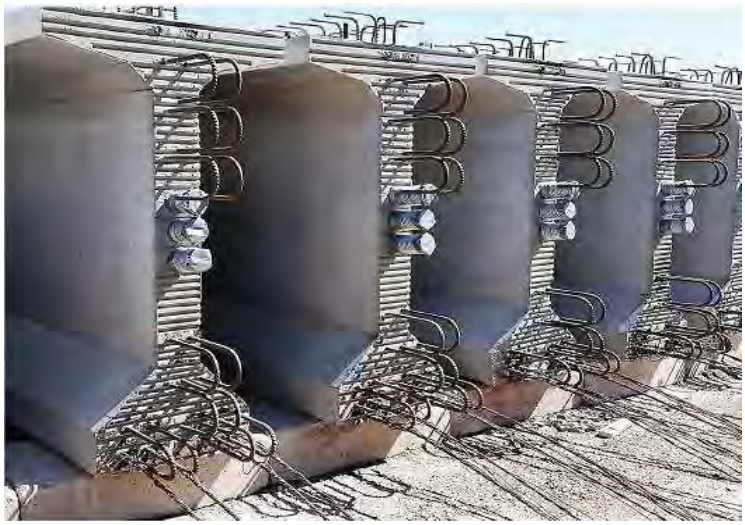

Figure 11 – PT Ducts and Precast Girder End Treatment

To provide similar structural behavior between the precast girder segments and the CIP box girder segments, the precast girder deck concrete was placed prior to post-tensioning of the superstructure. Erection of the precast girders coincided with the construction of the CIP box girder webs. This allowed

for simultaneous construction to reduce overall construction time and ensured proper alignment of the PT ducts.

Figure 12 – Setting Girders at Span 2

A two-foot splice closure pour was provided between the ends of the precast girders and the ends of the CIP box girder webs to allow for the splicing of the PT ducts. This end diaphragm also accommodated the end rotation of the precast girders resulting from the deck placement.

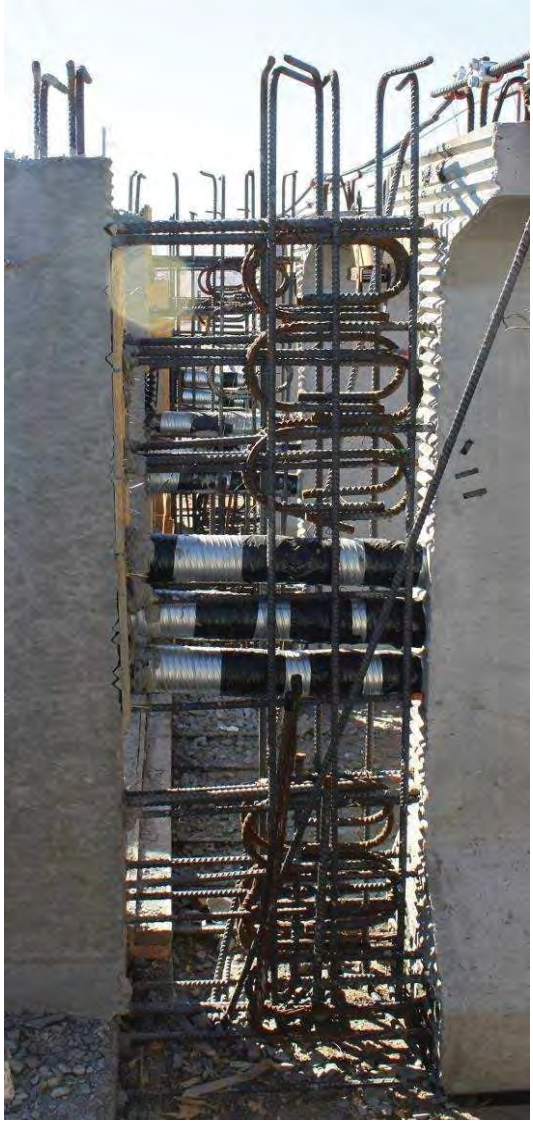

Figure 13 – Splice Closure Pour at Interior and Exterior

The superstructure was post-tensioned once the deck and splice closure pour concrete reached strength.

This tied the precast and CIP superstructure elements together to provide continuity, mimic the structural behavior of the existing bridge and carry the applied live loads.

At the abutment end of each precast girder of Spans 1 and 4, an 8’-6” long by 2’-4” wide end block was created by flaring the 8” web out to match the width of the bottom flange of the girder. The end block accommodated the post-tensioning anchorage hardware and provided the additional beam cross-section needed to resist post-tensioning forces.

Figure 14 – PT Anchors at Precast Girder End Block (Photo by Jason Kuck, Tpac)

PRECAST SAVES TIME AND MONEY

The existing bridge was widened about 120 feet, this equates to approximately 64,000 square feet of bridge deck. The spliced-girder hybrid bridge option with precast girder segments of 82 feet, 120 feet and

91 feet were erected in Spans 1, 2 and 4 respectively. Prior to being post-tensioned with the CIP pier tables, the precast girders were supported on temporary shoring about five feet from each end. Using precast girders eliminated approximately 265 linear feet of falsework or approximately 31,800 square feet of falsework with a net savings of over $954,000.

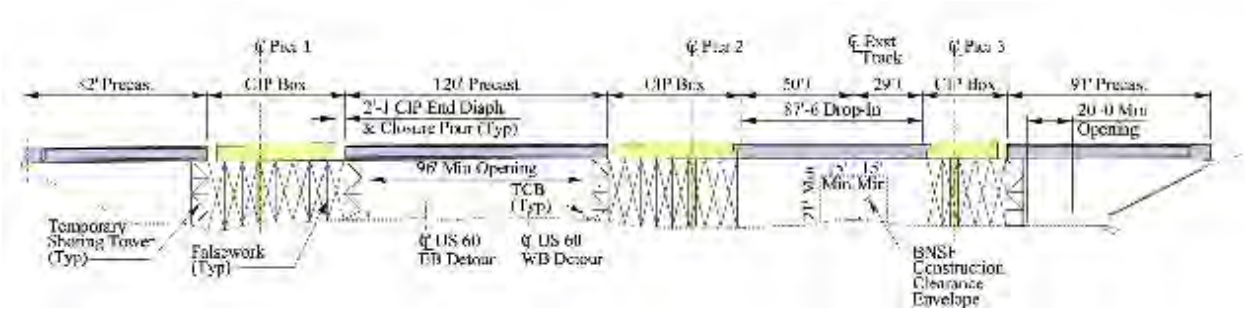

Figure 15 – Falsework Layout

In addition to the tangible monetary savings associated with eliminating falsework, another benefit of using precast girders was time savings. The man hours required to erect and break down almost 32,000

square feet of falsework could be directed toward other activities. There was also a time savings associated with the concurrent construction of different bridge elements. Portions of the superstructure were no longer dependent on the completion of the falsework, thus allowing for the simultaneous construction of bridge substructure elements and the fabrication of precast girders. The CMAR estimated that the construction schedule was accelerated seven weeks by using precast girders in place of CIP concrete.

PRECAST OPENS THE WAY

A crucial project requirement was maintenance of traffic on US60 during bridge construction. CIP construction on falsework over traffic is invariably disruptive to traffic below. Detouring US60 traffic was not practical and maintaining four lanes of traffic would have required large openings through any falsework. Two separate falsework openings would have been likely, one for each direction of US60 traffic. Using fixed falsework openings would have eliminated any possibility of shifting traffic to accommodate various construction phases.

Figure 16 – Traffic Opening at Span 2

At Span 2, a 120-foot long precast girder was spliced with 37-foot long CIP pier tables. Prior to posttensioning the superstructure, the precast girders were supported on shoring towers. This created an available clear opening of almost 100 feet. This large opening, with high vertical clearance, easily accommodated all traffic with two lanes in each direction plus a turn lane; it also provided flexibility during construction which permitted traffic shifts to accommodate phasing and improved public safety.

CONCLUSION

The US60/SR303L bridge widening demonstrated that the “obvious” approach, widen “in-kind”, isn’t necessarily the best approach. Widening the existing bridge using an identical superstructure type would have been simple from a design standpoint but wasn’t the best solution to meet the project goals. This innovative hybrid approach used relatively simple locally familiar construction methods and materials consisting of conventional, locally available precast/prestressed AASHTO girders combined with posttensioned and reinforced concrete structural elements.

The hybrid spliced precast concrete girder approach provided compatible structural behavior to the maximum practicable extent. The widening reflects the existing bridge span configuration, structure depth, hinge locations, support configurations and PT jacking points. This approach also reduced falsework requirements, traffic disruptions, construction costs and time. Construction of the widening began in December of 2014 and is expected to be completed by the end of November 2015. The estimated construction cost is $7,300,000.

G-16: DESIGN OF ABC PROJECTS FOR SERVICE LIFE

EXPERIMENTAL TESTING OF UHPC JOINTS OF PRECAST CONCRETE BRIDGE DECK

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INTRODUCTION

Efficiency of composite bridges may be increased if the concrete slab is made of precast elements. The construction time can be shorter when modern technology and advanced materials are used. Favorable material properties of UHPC [1,3], especially high bond stress, allow for a very significant reduction of the anchorage length of reinforcement. The dimensions of joints of precast concrete elements may be optimized, i.e. significantly reduced. It leads to application of small amount of UHPC, which may result in a cheaper design e.g. at precast concrete composite bridges. Another reason of using UHPC as a material of joints is its capability to transfer local high stress concentrations. The number of shear connectors may be significantly reduced provided their structure is adapted to the anchorage in UHPC. It is possible to design shear connectors locally in groups instead of continuously distributed along the entire length of the top flange of a steel girder. The experience from already erected structures around the world shows that UHPC, used as a material of joints, may be a very good alternative, which results in excellent structural performance, increased durability of joints and makes acceleration of bridge construction possible[1,2].

BOND OF REINFORCEMENT IN UHPC

The experimental verification of bond properties of UHPC showed that the anchoring length can be significantly reduced. The tests were carried out on using bars of the diameter 12, 16 and 20 mm according to the RILEM recommendation with the anchorage length of 5 diameters. All tests failed in steel and no bond failure was observed. Then the anchorage length was reduced to 4, 3 and 2 diameters. The anchorage length of 4 diameters was found as sufficient, and the specimens with anchorage length of 2 diameters failed by bond. The length of 3 diameters was found as a limiting anchorage length.

INITIAL EXPERIMENTS ON UHPC JOINTS

The models of the joints of precast concrete slabs made of UHPC were subjected to the bending tests. The two arrangement of reinforcement were designed. The joint was only 200 mm wide which allowed for sufficient length for anchorage of the steel bars of precast slabs because of use of UHPC. The scheme of the test is plotted in Fig. 1. A typical failure is illustrated in Fig. 2.

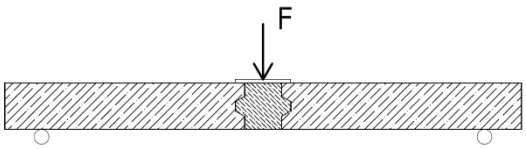


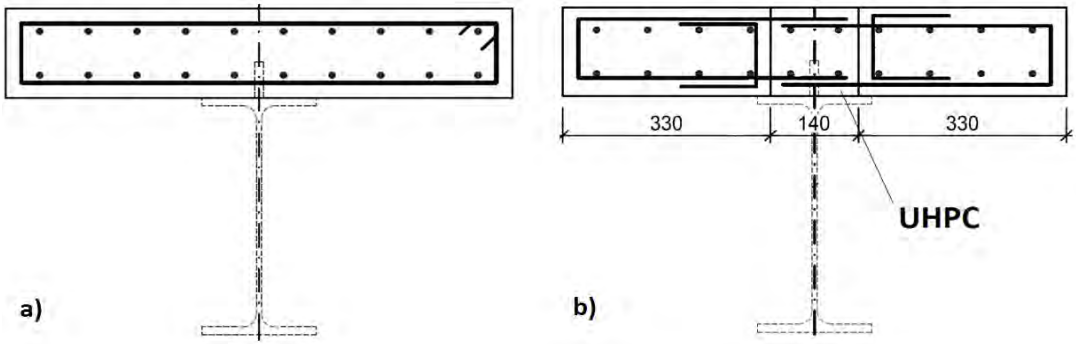
Fig.1: Scheme of the test

Fig.2: Typical cracking at the failure

The 6 tests of the joint were carried out. Under the serviceability load the cracks appeared only in ordinary concrete or at the border between ordinary concrete. Their width was in majority of specimens about 0.15 – 0.25 mm, which complies with requirements of codes. The ultimate load was slightly higher than expected, and again no cracks appeared in the UHPC. The existing cracks originated at lower load levels extended until complete failure of models. The experiments showed that the concept of the UHPC joints of precast elements works well and can be used in structures.

COMPOSITE BEAMS

Excellent results of experiments of UHPC joints of precast deck elements led to design experimental model of UHPC connection between steel girder and precast concrete deck. This experiment intends to verify the performance of the joint closer to its reality. The joint is subjected to transversal bending, to longitudinal shear between concrete and steel and to longitudinal bending. The shear studs are the most usual shear connectors. Such dimensions are not suitable for their application in the UHPC. The suitable connectors should be shorter and their diameter should be larger than of those which are available on the market. Therefore it was decided to replace the shear studs by a perforated steel sheet working as a shear connection, which may be designed for the purpose of the experiment in a more suitable way. The loading is applied by two forces acting on the cantilever on the concrete slab outside of the axis of the steel beam. The load was distributed along the length of about 1 m, in order to induce a transversal bending moment in the concrete slab. The scheme is shown in Fig. 3.

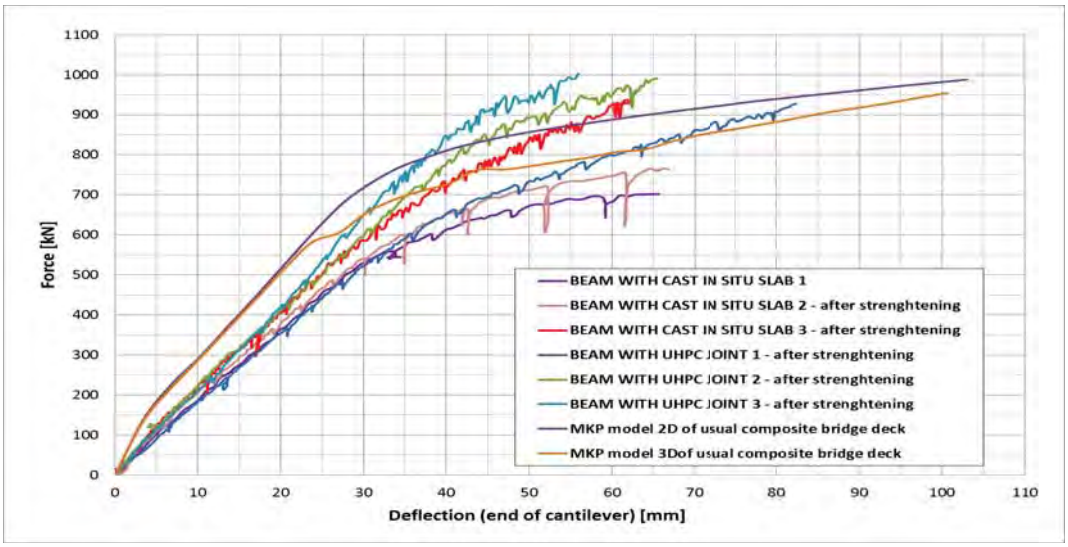


Cross-section of the beams a) cast in situ slab (type 1), b) precast slab with the joint (type2)

Two types of composite beams are examined. The first (beam 1) was made of ordinary concrete without the joint. It represents usual cast in situ composite bridge deck. The second type of composite beam (beam 2) has a two parts of ordinary concrete slab with the joint filled by UHPC and represents precast concrete deck. The perforated steel sheet design was different for each type of connection. In ordinary concrete perforated sheet was welded along the entire length of cantilever and in the UHPC joint small parts of sheets were locally welded in a cantilever part. In the left part simulating fixed end of the beam, the perforated steel sheet is identical at individual beams, since the failure in shear should be avoided in this part.

The results prove that welded perforated steel sheet is a sufficient connector. Almost no relative slip between the concrete slab and the steel beam was measured. The ultimate load of third specimen was about 932 kN. Extensive numerical analyses were done for verification of experimental results.

After the first part of tests of usual composite beams, the composite beams with UHPC joints were prepared for testing. Test procedure was the same as in previous test. The results prove that small parts of perforated sheets welded on the top of the steel flange and connected by UHPC are still a sufficient connector. Under loading about 800-900 kN the ultimate deflection between concrete and steel girder was about 0,8mm at the beginning of cantilever part. Almost no relative slip between the concrete slab and the steel beam was measured until force about 400kN. The maximum ultimate load of all specimens was about 1001 kN. The test were stopped because of very high loading force and because loading capacity of all composite beams with UHPC joints has reached ultimate loading capacity of usual composite beams tested in the first part of experiment. All L-D diagrams are plotted in fig 4.



Comparison of load deflection diagrams of experiment and numerical analysis

CONCLUSIONS

The first applications of UHPC can be found in the Czech Republic in both, building and transport structures. The first experiments which have been executed at the CTU in Prague show that also joints may be a field, where UHPC can find its application in load carrying structures. The high strength of UHPC in compression as well as in tension predetermines UHPC to be used in areas with high stress concentrations. The joints are exactly those locations where the stress may be high and where the interaction of stress in different directions takes place. The excellent bond between steel and UHPC makes it possible to reduce the size of joints, which is appreciated from the technological as well as from the economical point of views.The experimental programs which are being executed follow two purposes:

1. A practical verification of an actual performance of a complex structural detail. Without an experiment it would be very encouraging to rely only on numerical analysis at such complex structural details.

2. From experimental results the necessary input data for numerical analyses will be received.

All results of composite beams with precast bridge deck connected with UHPC filled in the joint fulfilled expectations and the ultimate forces were same or higher than composite beams with in situ cast bridge deck. Almost no deflection between concrete deck and steel girder was observed at first part of specimens (composite beams 1) and also at second part of specimens with reduced perforated steel sheet UHPC joints. Experiments have shown that UHPC is very sufficient, modern material for complicated details solutions.

ACKNOWLEDGEMENT

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EXPERIMENTAL TESTING OF UHPC JOINTS OF PRECAST CONCRETE BRIDGE DECK

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ABSTRACT

Precast concrete elements are usually connected using joints filled with ordinary concrete. The joint must allow for a reliable transfer of tensile forces in reinforcement. If the joint is filled by UHPC, the tensile forces in reinforcement may be efficiently transferred. Experimental research has proven that the anchorage length of the reinforcement in UHPC is significantly shorter than that in ordinary concrete. It allows for designing a very short overlaps of bars in UHPC. Joint made from UHPC may be rather small and no mechanical connection of steel is necessary. The results of tests will be presented in this paper.

INTRODUCTION

Efficiency of composite bridges may be increased if the concrete slab is made of precast elements. The construction time can be shorter when modern technology and advanced materials are used. Favorable material properties of UHPC [1,3], especially high bond stress, allow for a very significant reduction of the anchorage length of reinforcement. The dimensions of joints of precast concrete elements may be optimized, i.e. significantly reduced. It leads to application of small amount of UHPC, which may result in a cheaper design e.g. at precast concrete composite bridges. Another reason of using UHPC as a material of joints is its capability to transfer local high stress concentrations. The number of shear connectors may be significantly reduced provided their structure is adapted to the anchorage in UHPC. It is possible to design shear connectors locally in groups instead of continuously distributed along the entire length of the top flange of a steel girder. The experience from already erected structures around the world shows that UHPC, used as a material of joints, may be a very good alternative, which results in excellent structural performance, increased durability of joints and makes acceleration of bridge construction possible[1,2].

BOND OF REINFORCEMENT IN UHPC

Extensive research program of bond between reinforcement and concrete and especially UHPC was carried out in Klokner Institute, CTU in Prague. Bond is one of the basic parameters for design of concrete structures, since it influences especially the anchorage length and then the design of the structure. Steel reinforcement in concrete requires designing the anchorage length for reliable transfer of the tensile force in steel bars into concrete. The shear stress between reinforcement and concrete and its distribution along the anchorage length of the reinforcement are the key parameters for the design of advanced structures made of UHPC with emphasis on their reliability and long-term durability. For evaluating the average shear stress on the boundary of the steel bar and concrete the RILEM RC6 recommendation was used. The steel bar with anchorage length 5 x diameters is embedded in the cube with dimensions 200 x 200 x 200 mm and then pulled out. In order to simplify the evaluation of experiments the assumption of uniformly distributed stress is accepted. The average shear stress is given by the ratio of tensile force in the reinforcement and contact area between steel bar and concrete. The experiments, which were carried out, had two parts. The results of the first part did not lead to failure of bond, in the second part the conditions were modified, in order to get the expected failure mode [3,4]. In the first part of the experiment, the three diameters of steel bars embedded in UHPC were tested (12, 16 and 20 mm). For comparison the same tests were carried out using the cubes made of ordinary concrete of the class C30/37.

The failure of steel bars was observed at all specimens made of UHPC, while the failure of bond was observed at all specimens made of ordinary concrete. The average bond stress was significantly lower than that measured in the specimens made of UHPC. The results are plotted in fig.1.

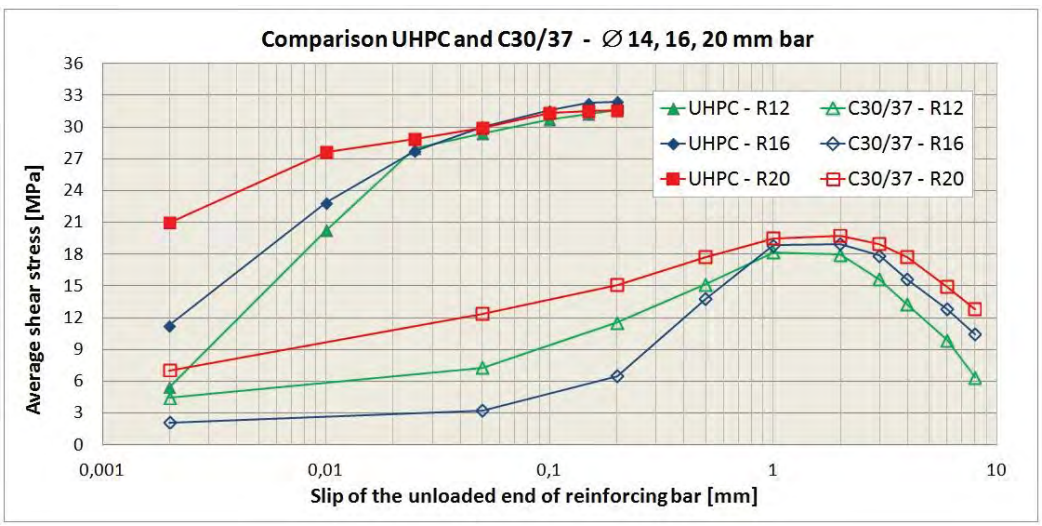

Figure1: Comparison of the bond behaviour of UHPC and C30/37 - different bar diameters

The second series of tests was focused on the reduction of the anchoring length to 4 diameters, 3 diameters and 2 diameters of the steel bar. The balance between tensile strength of the bar and the bond capacity at the reduced anchoring length was searched. These tests showed that anchorage length of 4 diameters is sufficient (the failure of steel bar was similar to those with the anchorage length of 5 diameters). The failure in bond appeared when the anchorage length was reduced to 2 diameters of steel bar. The highest average shear stress was reached by specimens with the anchorage length of 3 diameters. At this value of anchorage length the largest ratio of tensile strength, which was close to the strength of steel and concrete reinforcement contact area, was observed. It can be concluded that the using of UHPC was significantly (about 2.5 times) increased the maximum average bond stress compared to normal concrete C30/37.

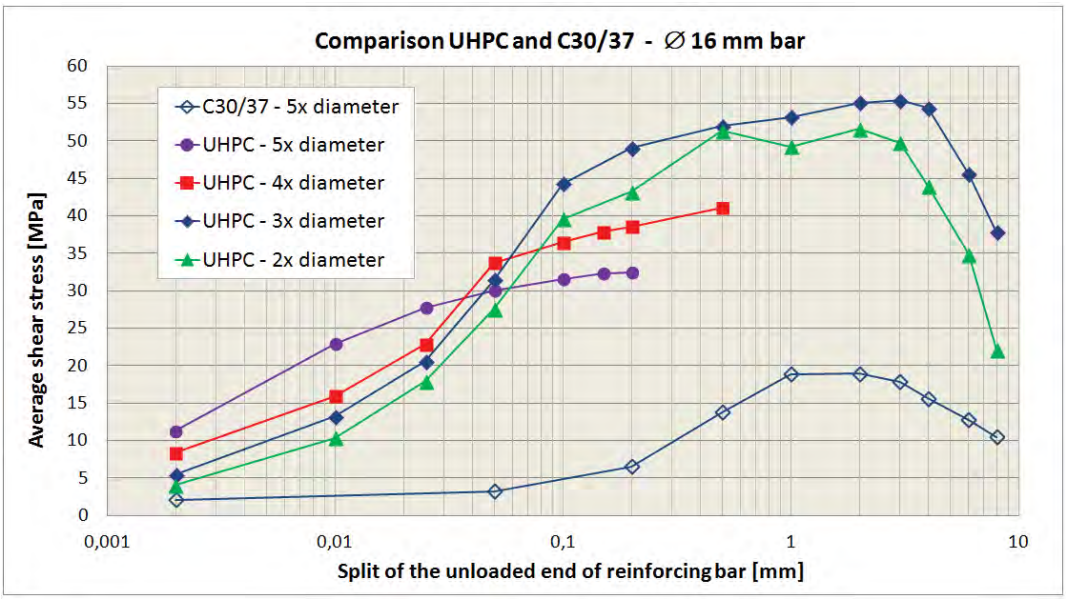

Figure2: Comparison of the bond of UHPC and C30/37 - different anchorage length

INITIAL EXPERIMENTS ON UHPC JOINTS

The research is focused on the performance of the precast concrete deck of a steel concrete composite bridge. The precast elements of the slab will have the longitudinal joints above the steel beams and transversal joints which will be perpendicular to the steel beams. The longitudinal joints are subjected to a large bending moment in transversal direction. Additional to that, the stud connectors welded to the top flange of the steel beam will be embedded in the joint, which results in additional stress in the joint. The application of UHPC as a filling of the joint has significant advantages. 1. The high strength of UHPC allows for reduction of the width of the joint, because the anchoring length of steel bars can be reduced (as it was already shown). 2. The smaller dimensions of the joint result in reduction of the costs, since the

UHPC is expensive. 3. The high strength of UHPC will contribute to the load carrying capacity of the shear stud connectors. 4. The joint can be easily located only above the top flange of the steel beam, i.e. no additional formwork is necessary. The precast slabs can be supported on the edges of the steel flange and still there is enough space for the joint. In the first step only the bending behaviour of the joint was experimentally verified.

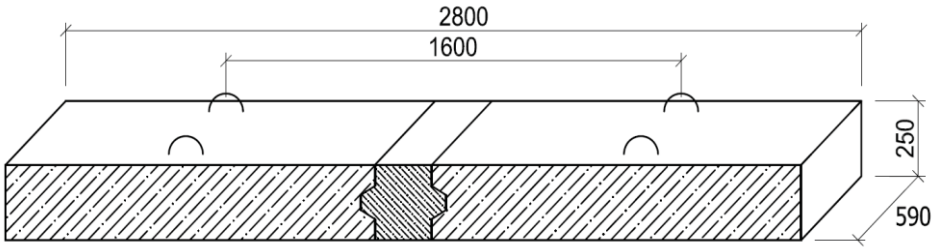

Figure3: Scheme of the specimen with dimensions

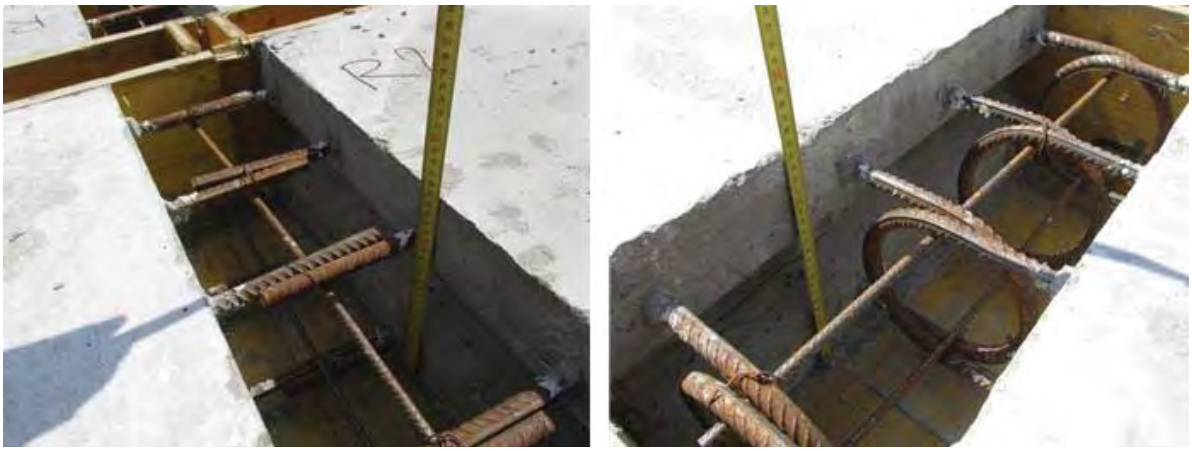

Figure4: Straight bars in UHPC joint (type R) and bars in the loop (type S)

Design and fabrication of the UHPC joints of the precast elements

The thickness of the bridge deck slab of 250 mm was assumed, since it corresponds to the realistic bridge design. The model was designed 0.6 m wide and 2.8 m long. The width of the joint was only 200 mm, which allows for a comfortable placement of the reinforcement and also the volume of the joint is not large (Fig. 3). The two arrangements of the reinforcement of the joint were tested. The first arrangement (type R) had only straight bars coming out from the precast slabs (Fig. 4 left). The second arrangement (type S) had the loops made of reinforcing bars which overlapped in the joint (Fig. 4 right). The profile of the steel was identical in individual alternatives (14 mm).The precast elements – model of deck panels were made from ordinary concrete C40/50. After hardening of the panels the joint was cast using UHPC. The UHPC has a cylinder concrete strength over 150 MPa, the flexural strength about 18 MPa and it

contains about 2% of short high strength fibres. The self-compacting UHPC was used; therefore no additional compacting was necessary. This is also important for future practical applications, since the compacting would not be easy due to the relatively dense reinforcement of the joint. The surface of UHPC was left without any additional smoothing and it was carefully treated with water and covered with PE foil to prevent evaporation. After hardening, there were not found any cracks in the UHPC joint resulting from shrinkage strains.

Test procedure

The testing procedure represents the transversal bending of the bridge deck above the flange of the steel beam. The test setup is arranged in the opposite position, i.e. the reaction of the flange is represented by a force *F* acting in the middle of the model (Fig. 5). The top flange of the beam is simulated by the steel plate under the loading force.The experiment involved testing of 6 specimens. 3 specimens had the reinforcement of the joint of type R and the other 3 specimens were reinforced by the type S. The other parameters were identical.

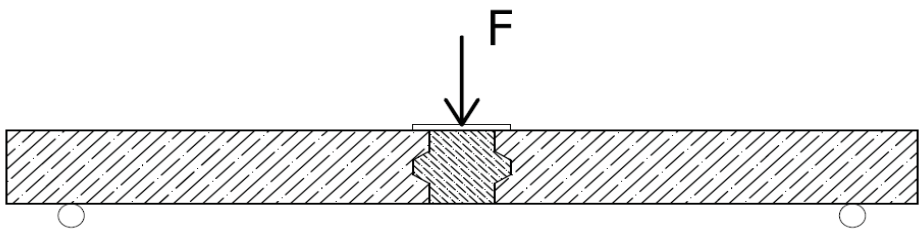

Figure5: Scheme of the test

The deflection was measured at the midspan at two points and also the displacements at the supports were monitored. The potentiometric displacement sensors were used. The specimens were loaded in 5 cycles up to the level of the serviceability load (about 50% of estimated ultimate load). Then the load was increased until failure. The loading process was controlled by force when loading up to the serviceability level and by deflection growth, when loading until failure. The test was terminated after a significant decrease of load forces or if deflections grew at constant load. The objective of the tests was twofold: 1. Load – deflection diagram and 2. Observation of cracks in relation to the load level and their position in relation to the joint.

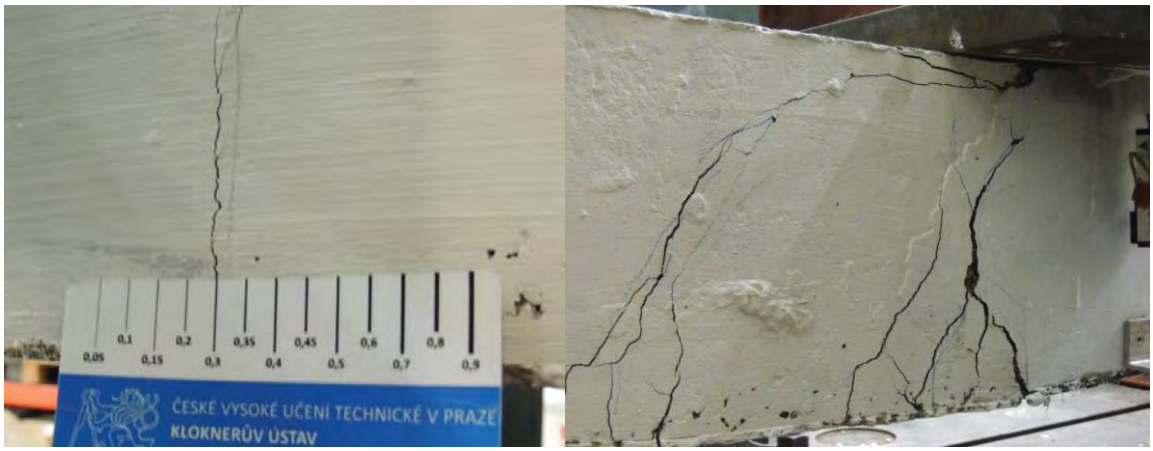

Figure6: Measurement of crack width, expansion of main cracks

Results

Similar pattern was observed at all tests. The first hair cracks appeared at the load level about 40 kN. At the estimated serviceability load, the crack width was about 0.15 to 0.25 in average in dependence on the reinforcement (Tab.1). Such result would be completely acceptable for a characteristic load combination in SLS. No cracks were observed at the UHPC joint, they were on the boundary between the two concretes or at the precast part of the model (Fig. 6).

Test results

Number of panel	Loading force - first crack [kN]	Maximum crack width under serviceability load 75 kN [mm]	Ultimate loading force [kN]
R1	40,0	0,2	167,5
R2	35,0	0,25	168,5
R3	38,0	0,3	177,2
Average of R	37,7	0,25	171,1
S1	45,0	0,1	164,1
S2	36,0	0,15	157,4
S3	40,0	0,15	159,6
Average of S	40,3	0,13	160,4
Average of all specimens	39,0	0,19	165,7

Collapse of the model was achieved at the load level slightly higher than expected (in the range 160 kN to 170 kN, in dependence on the reinforcement type). The main failure crack was located either on the boundary between UHPC and ordinary concrete (at the loop reinforcement – type S) or in ordinary concrete (at the majority of specimens with straight reinforcement – type R). The load displacement diagrams of all specimens are plotted in Fig. 7.

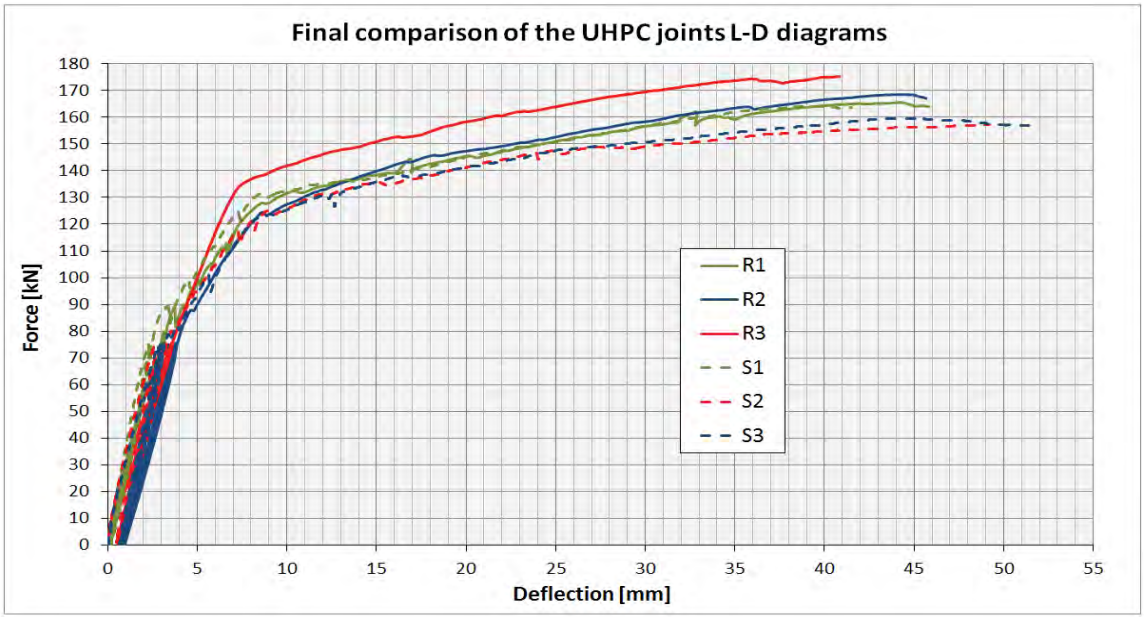

Figure7: Comparison of the UHPC joints L-L-D diagrams

COMPOSITE BEAMS

Excellent results of experiments of UHPC joints of precast deck elements led to design experimental model of UHPC connection between steel girder and precast concrete deck. This experiment intends to verify the performance of the joint closer to its reality. The joint is subjected to transversal bending, to longitudinal shear between concrete and steel and to longitudinal bending. The concrete over the steel beam resists in the supports areas of the continuous bridge to biaxial bending and to shear. In order to fit the performance of the model to the actual performance of the bridge slab, the cantilevers were chosen as appropriate models, which satisfy the requirements on tests and the dimensions of which will not be too large. Their dimensions are smaller than that of the real bridge. Composite beams are 3.8 m long, concrete deck is 150 mm thick and the steel girder is 400 mm deep. The shear studs are the most usual shear connectors. They are designed for application in ordinary concrete. Their dimensions are accommodated to the failure ratio between concrete and steel. I.e. the pull out force defining the concrete failure and the load carrying capacity of the welded joint of the stud to the steel beam are balanced. The usual studs are therefore rather long (corresponding to the concrete strength) and thin. Such dimensions are not suitable for their application in the UHPC. The suitable connectors should be shorter and their diameter should be larger than of those which are available on the market. Therefore it was decided to replace the shear studs by a perforated steel sheet working as a shear connection, which may be designed for the purpose of the experiment in a more suitable way. The loading is applied by two forces acting on the cantilever on the concrete slab outside of the axis of the steel beam. The load was distributed along the length of about 1 m, in order to induce a transversal bending moment in the concrete slab. The slip between concrete deck and

steel girder, the deflection of the end of the cantilever and the edges of concrete deck under loading cylinders were measured. The scheme is shown in Fig. 8.

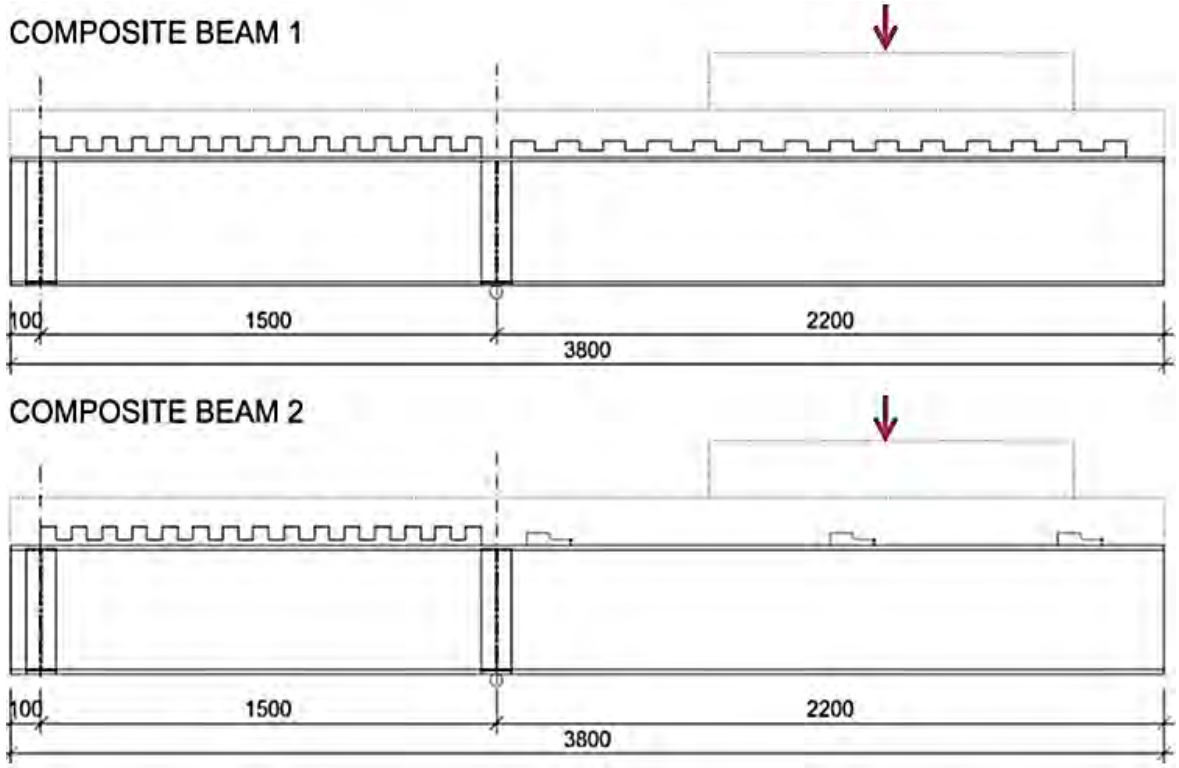

Figure8: Longitudinal scheme of two types of composite beams

Two types of composite beams are examined. The first (beam 1) was made of ordinary concrete without the joint. It represents usual cast in situ composite bridge deck. The second type of composite beam (beam 2) has a two parts of ordinary concrete slab with the joint filled by UHPC and represents precast concrete deck. The perforated steel sheet design was different for each type of connection. In ordinary concrete perforated sheet was welded along the entire length of cantilever and in the UHPC joint small parts of sheets were locally welded in a cantilever part. In the left part simulating fixed end of the beam, the perforated steel sheet is identical at individual beams, since the failure in shear should be avoided in this part. Prepared specimen for test is shown in Fig. 10.

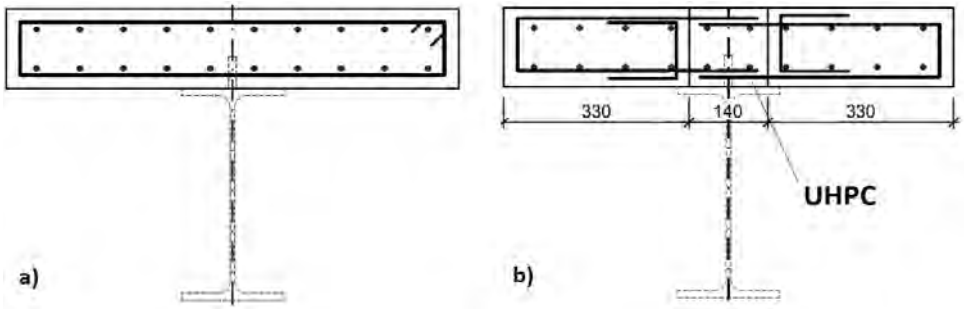

Figure9: Cross-section of the beams a) cast in situ slab (type 1), b) precast slab with the joint (type2)

At first the composite beams with cast in situ slab were tested. The results prove that welded perforated steel sheet is a sufficient connector. Almost no relative slip between the concrete slab and the steel beam was measured. The ultimate load of third specimen was about 932 kN. Extensive numerical analyses were done for verification of experimental results. The numerical analysis is still continuing but some results are plotted in Fig. 11.

Figure 10: Composite beam with precast bridge deck with UHPC joint

After the first part of tests of usual composite beams, the composite beams with UHPC joints were prepared for testing (Fig. 10). Test procedure was the same as in previous test. The results prove that small parts of perforated sheets welded on the top of the steel flange and connected by UHPC are still a sufficient connector. Under loading about 800-900 kN the ultimate deflection between concrete and steel girder was about 0,8mm at the beginning of cantilever part. Almost no relative slip between the concrete slab and the steel beam was measured until force about 400kN. The maximum ultimate load of all

specimens was about 1001 kN. The test were stopped because of very high loading force and because loading capacity of all composite beams with UHPC joints has reached ultimate loading capacity of usual composite beams tested in the first part of experiment. All L-D diagrams are plotted in fig 11.

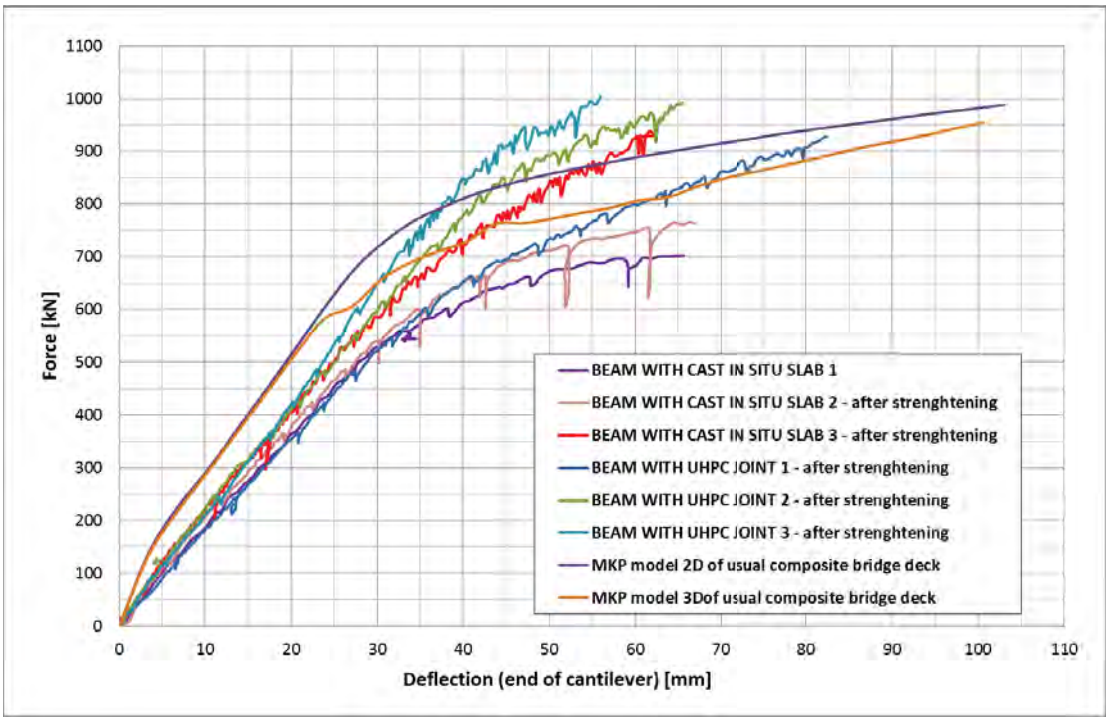

Figure11: Comparison of load deflection diagrams of experiment and numerical analysis

CONCLUSIONS

The first applications of UHPC can be found in the Czech Republic in both, building and transport structures. The first experiments which have been executed at the CTU in Prague show that also joints may be a field, where UHPC can find its application in load carrying structures. The high strength of UHPC in compression as well as in tension predetermines UHPC to be used in areas with high stress concentrations. The bond tests showed that the anchoring length of the reinforcing bars in the UHPC can be significantly reduced in comparison with that used in ordinary concrete. The anchoring length of only 4 diameters appeared as completely sufficient, the bars failed in tension, while the bond was not subjected to any damage. Such result was optimistic for design of the reinforcement of the UHPC joint of precast elements of the bridge deck of the steel concrete composite bridge. The 6 tests of the joint were carried out. Under the serviceability load the cracks appeared only in ordinary concrete or at the border between ordinary concrete. Their width was in majority of specimens about 0.15 – 0.25 mm, which complies with requirements of codes. The ultimate load was slightly higher than expected, and again no cracks appeared in the UHPC. The existing cracks originated at lower load levels extended until complete failure of

models. The experiments showed that the concept of the UHPC joints of precast elements works well and can be used in structures. At the moment the reference specimens with the slab made of ordinary concrete were tested and satisfactory function of the shear connection using a perforated steel sheet has been found. The joints are exactly those locations where the stress may be high and where the interaction of stress in different directions takes place. The excellent bond between steel and UHPC makes it possible to reduce the size of joints, which is appreciated from the technological as well as from the economical point of views.

The experimental programs which are being executed follow two purposes:

1. A practical verification of an actual performance of a complex structural detail. Without an experiment it would be very encouraging to rely only on numerical analysis at such complex structural details.

2. From experimental results the necessary input data for numerical analyses will be received.

Excellent results of experiment of UHPC joint of precast deck elements led to design experimental model of UHPC connection between steel girder and precast concrete deck. This experiment includes longitudinal, transverse and shear stress and describes functionality of real detail of steel concrete composite bridge structure. All results of composite beams with precast bridge deck connected with UHPC filled in the joint fulfilled expectations and the ultimate forces were same or higher than composite beams with in situ cast bridge deck. Almost no deflection between concrete deck and steel girder was observed at first part of specimens (composite beams 1) and also at second part of specimens with reduced perforated steel sheet UHPC joints. Experiments have shown that UHPC is very sufficient, modern material for complicated details solutions. In extensive experiment program the favorable material properties has been proved and has been shown that UHPC should be used in many difficult construction details.

ACKNOWLEDGEMENT

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MODERN NON-BITUMINOUS FLEXIBLE PLUG EXPANSION JOINTS – MINIMIZING NOISE, MAXIMIZING DRIVER COMFORT AND ACCELERATING BRIDGE MAINTENANCE

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ABSTRACT

A new, much improved type of flexible plug expansion joint has been developed, with a polyurethane surface, which offers a number of substantial advantages over the traditional bituminous type. The *Polyflex®Advanced* expansion joint offers all the benefits of the asphaltic plug joint – including smooth, safe, low-noise surface, great adaptability and easy installation. However, it overcomes the numerous disadvantages and challenges that have always plagued asphaltic plug joints. It offers enormously improved strength, elasticity and durability, resulting in much less maintenance and far more reliable watertightness. Installation is far easier and less prone to error, with the two-component compound being mixed at ambient temperatures. For these reasons and others, this type of joint should be considered for use in bridge construction and, in particular, in bridge maintenance.

INTRODUCTION

Flexible plug expansion joints, which create a completely closed, absolutely flat driving surface right across a structure's movement gap, offer a number of benefits over other small-movement expansion joint types. The continuous, flexible surface results in unsurpassed driver comfort and extremely low noise under traffic, while also eliminating discomfort and safety risks for pedestrians and cyclists. Furthermore, the way the joints are constructed, by pouring freshly mixed material in situ, facilitates transport and handling and makes expansion joints installable in sections, lane by lane, with any desired shape or longitudinal profile (e.g. with intersections or upstands).

However, flexible plug expansion joints made from traditional bituminous materials have long been plagued with durability problems, not performing as well, long term, as joints manufactured primarily from steel in factory conditions. Continuous dynamic loading and braking/acceleration forces from vehicle wheels, year after year, cause stresses in the material, resulting in cracking, loss of watertightness and general deterioration – impacts that would only be exacerbated by improper preparation on site and incorrect temperature during installation (typically approx. 180 °C / 350 °F required).

To overcome these shortcomings while retaining the aforementioned benefits, the design of the flexible plug expansion joint has been optimized, utilizing superior (non-bituminous) materials and incorporating improved support and connection details. The result – the *Polyflex®Advanced* expansion joint – is described below.

HISTORY AND BACKGROUND

Traditional bitumen-based flexible plug expansion joint materials suffer from several disadvantages. At low temperatures, for instance, the material used is generally very stiff, causing de-bonding and resulting in leaking, while at high temperatures, the material becomes weak and tends to deform plastically. Inconsistent quality due to improper mixing and incorrect temperature during installation (high temperatures required) also frequently cause problems. As a result of such disadvantages, asphaltic plug joints are subject to various limitations. For example, they should generally not be installed in highways and locations with frequent acceleration and braking, such as in the vicinity of bus stops, traffic lights etc., and they should not be used in railway bridges under stone ballast. And in general, the functionality and durability of asphaltic plug joints has often been found from experience to be unreliable.

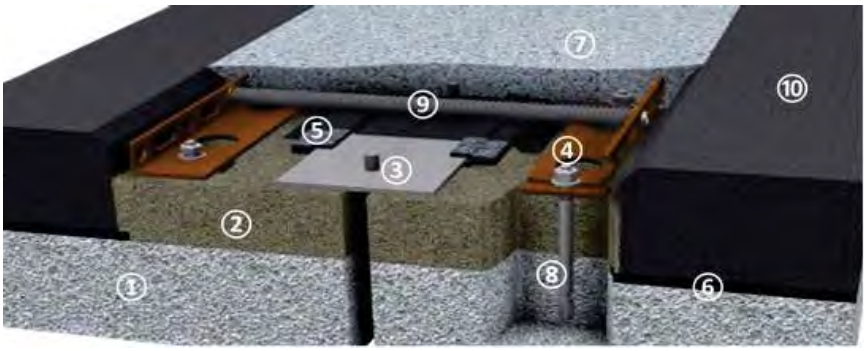
In Switzerland, investigations carried out in cooperation with EMPA (the Swiss Federal Laboratories for Materials Science and Technology) showed that the bitumen quality being used for such joints varied considerably, with substantial effects on joint functionality and durability. Small changes in the chemical composition of the raw materials led to big reductions in expansion joint quality. As a result, construction project clients and expansion joint manufacturers became increasingly concerned about the ongoing suitability of the systems for use. A demand developed for a flexible plug system using plug material that could be produced by the expansion joint supplier, or that could at least be acquired from a materials supplier in the required quality.

DESIGN AND CHARACTERISTICS OF THE IMPROVED PU-BASED FLEXIBLE PLUG JOINT

Instead of the bituminous material traditionally used to form the driving surface of flexible plug expansion joints, the *Polyflex®Advanced* expansion joint uses a specially selected, solvent free, highly durable polyurethane (PU) material.

The PU material originally used has a long history of use as waterproofing for roofs and has been constantly improved over the years. The material has shown test values of 650% elongation before breaking (compared to 350-400 % for standard rubber), which enhances durability and makes the material an ideal choice for use in expansion joint systems.

With perforated steel support elements incorporated in the design, the joint can withstand long-term traffic loading and braking and reaction forces while accommodating significant structure movements, at both very low and very high temperatures. Total movements of up to 100 mm (4 inches) have been accommodated in several countries on various projects in successful operation since 2007.



Main structure

1 Polymer concrete base
2 Bridge plate across gap
3 Perforated steel angle
4 Sponge rubber
5 Bridge waterproofing
6 Special PU material
7 Anchoring
8 Stabilizing bar if needed
9 Road surfacing

Illustration showing the main elements of a typical Polyflex®Advanced expansion joint

In addition to its exceptional elasticity, the special PU material used offers enormous tear resistance, with a tear strength of 20 N/mm². It typically has a tensile strength of 14 N/mm², a density 1.05 g/cm³ and a Shore A hardness of approximately 65. It is highly resistant to wear and environmental and chemical influences, and thus offers an exceptionally long lifespan. In fact, its service life is typically substantially longer than that of connecting roadway surface materials.

The joint is fully functional in the temperature range –50°C to 70°C (–58°F to 158°F) – a major improvement over asphaltic plug joints. It is also very versatile, with virtually any common joint shape possible – e.g. with upstands, skew angles and junctions (T-shaped and X-shaped junctions, etc.).

Intersections such as T-shape or X-shape possible

Upstands can be easily created

Installation is relatively easy, not only in comparison with traditional asphaltic plug joints but also compared with expansion joints of other types. With no large, heavy parts, lifting plant is not required, and the poured material adapts to suit the dimensions of the prepared recess. The two-component PU material is mixed from complete packing units at ambient temperatures, minimizing the risk of suboptimal mixing and installation. Processing is possible at temperatures from 5 °C to 35 °C (41 °F to 95 °F), virtually independent of humidity, and the joint can be driven over after only a few hours.

In the context of bridge maintenance, in particular – when the joint is installed to replace an existing one – the benefits of the joint's use are even more pronounced. The joint can typically be laid within the depth of a bridge's asphalt surfacing, avoiding the need to break out any concrete etc. With only minimal amounts of an existing structure to be removed, and quick installation and short material curing times, the new joint can be installed quickly, economically and reliably. The speed of installation (e.g. with a joint replaced during a night shift), with new joints being trafficable within just a few hours, minimizes impacts on traffic. If required, impacts on traffic can be further reduced by installing the new joint lane by lane – an approach that is fully supported by the joint's design. In phased installation, the already cured PU material of a previous stage is chemically reactivated by the fresh material, creating a high-strength bond. The same chemical reactivation of previously cured PU material also enables minor damage to an existing polyurethane joint to be easily repaired, simply by pouring fresh material onto the damaged area.

Summary of advantages and benefits

- Exceptional long working life, longer than adjacent surfaces
- Highest possible driver comfort
- No noise from crossing traffic due to surface that is flush with adjacent road
- Watertight
- Maintenance-free (no cleaning required)
- Quickly installed lane-by-lane with minimal traffic impacts, drivable after a few hours
- Installation within a wide temperature range (5°C to 35°C / 41°F to 95°F)

- Wear-resistant, no mechanical wear parts
- No rutting, high resistance to abrasion (e.g. from braking traffic)
- Damage to the joint can be easily repaired by reactivation of the PU material
- No recess for anchorage in structural concrete necessary
- Surfacing (asphalt or concrete) can be applied continuously before joint installation
- Any horizontal bend in the joint possible
- Any curb / sidewalk detail possible
- Low reaction forces
- Cold processing and easy handling with preset mixing ratio minimizes risk of mixing errors
- Resistant to environmental influences and acids, bases, chlorides, etc.
- Smooth surface ideal for pedestrian areas (e.g. in airports and railway stations)

INSTALLATION

The installation of a *Polyflex®Advanced* expansion joint to replace an existing joint is described below.

The recess is prepared by removing as much of the existing structure as is necessary to create the minimum space required to ensure an adequately strong, stable structure to which the polymer concrete base material can bond and transfer forces.

Cutting of continuous surfacing across the joint

Removal of existing joint / surfacing as required

The recess is then sandblasted as required to ensure proper adhesion of the expansion joint materials, and cleaned.

Where applicable, deck waterproofing membrane can be extended into the recess, enabling a watertight connection to be created.

Arrangement of ends of waterproofing membrane in recess

Where a suitable base must be created (in the absence of an appropriate surface following breaking out of the old joint), shuttering / formwork is then prepared to retain the fresh base material. This may simply take the form of a sheet of *Styrofoam*® or similar, placed in the bridge gap. A suitable primer is then used, if necessary, to ensure proper bonding and polymer concrete is poured to form the base.

Application of primer to substructure

Forming polymer concrete base (Styrofoam in gap)

The recommended *Robo*®*Flex* polymer concrete (if required) cures naturally, requiring only protection from the elements and from damage. Curing time depends on ambient temperature (at 15°C, approx. one hour). The supplied steel angles are anchored to the surface of the prepared surface at each side of the movement gap, and the supplied coverplate is placed across the gap, centred above it, as shown in the following photos.

Fixing of steel angles to polymer concrete base

Fixing of steel angles, and coverplate across gap

When all is prepared and confirmed, with the recess free of debris etc., the PU material can be poured and precisely levelled to the final level of the connecting surfacing.

Placing of PU material

Precise levelling of material to connecting surfaces

TESTING IN CONNECTION WITH AWARDING OF EUROPEAN TECHNICAL APPROVAL (ETA)

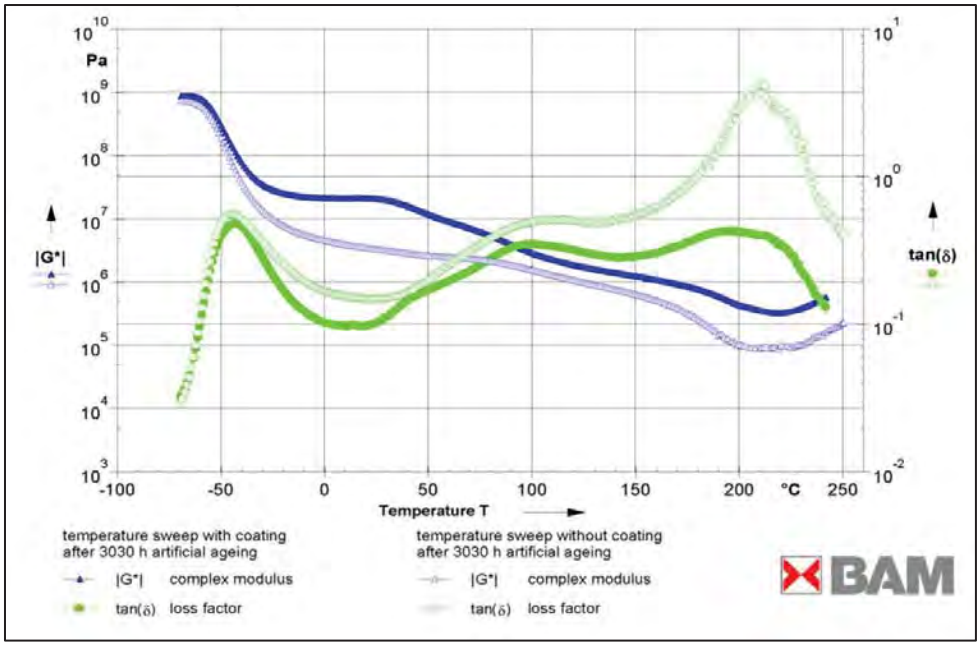
In connection with the awarding of a European Technical Approval, with validity across the European Union, extensive testing and certification was carried out by the *Bundesanstalt für Materialforschung und –prüfung (BAM)*, Berlin, by the *Prüfamt für Verkehrswegebau* of the Technical University of Munich (TUM), and by the MAPAG testing institute, Austria.

Testing of bond strength of the PU material

The tests included verifications of bond strength on various surfaces such as concrete, polymer-concrete, steel and asphalt. The recorded values were very high, even at low temperatures, demonstrating excellent resistance to de-bonding and thus also excellent resistance to leaking.

Assessment of ageing and temperature characteristics of the PU material

The ageing and temperature characteristics of the PU joint filling mixture were evaluated at the BAM institute in Berlin, after ageing for 3030 hours. The evaluation, based on ISO 4664, was carried out over a temperature range from -60°C to + 250°C. Both the complex modulus |G*| and the loss factor tan δ demonstrate very good performance for the declared temperature range of -40°C to +60°C.



Assessment of temperature characteristics of the PU joint filling mixture at the BAM institute, Berlin

Mechanical resistance testing

At the TUM institute in Munich, a full-scale assembled joint specimen, in the maximum opening position, was subjected to a test load of 150 kN via a pneumatic tyre. The contact pressure was 0.94 MPa, the temperature was 23°C and the specimen length was approximately 1 m. The test was carried out in accordance with the Austrian standard RVS and the appropriate European Technical Approval Guideline (draft ETAG 032, Part 3, Annex 3M Method a), and recorded deformation after loading and any subsequent recovery curve. In the test, deformations of max. 0.5 mm were recorded immediately after unloading, and within one hour of unloading, a complete elastic recovery of the surface had occurred. No damages or other changes to the surface were detected.

Mechanical resistance testing at the TUM institute, Munich

Fatigue resistance testing

A second full-scale joint specimen was then subjected to further testing in the same testing rig at TUM, Munich. The test involved repeated rolling over by a pneumatic wheel, at an elevated temperature of 45°C, in accordance with draft ETAG 032, Part 3, Annex 3M Method b. The contact pressure of the pneumatic tyre was 1.0 MPa, and the number of overpasses was 3030, with 30 of these executed with an additional 10% of horizontal load to simulate braking forces. After the test, no de-bonding or cracking was observed, and the test was passed.

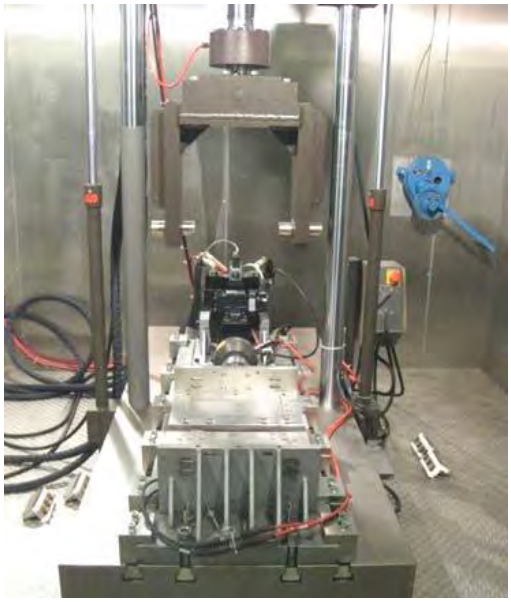
On the basis of experience in Europe with the same testing procedure for asphaltic plug joints and various national regulations, this successful high-temperature testing would support a 15-year service life categorization.

In addition, a standardized rutting test was carried out, at 60°C, in accordance with EN 12697-22. The following pictures show the enormous difference in performance between traditional asphaltic plug joint material and the *Polyflex®Advanced* material.

Comparison of common asphaltic plug joint material and the new Polyflex® Advanced flexible plug joint material after rutting test acc. to EN 12697-22 at 60°C (Left: Common asphalt plug material after 100 cycles at 60°C. Right: Polyflex® Advanced after 30,000 cycles at 60°)

Movement capacity testing

To evaluate the movement capacity of the full-scale joint specimen, a test was performed at the BAM institute in Berlin, in accordance with draft ETAG 032, Part 3, Annex 3N. The complete declared movement range, from maximum elongation to maximum compression, was tested, with temperature varying synchronously to the relevant deformation state between -40°C and +60°C. During the test, reaction forces and deformations were recorded.

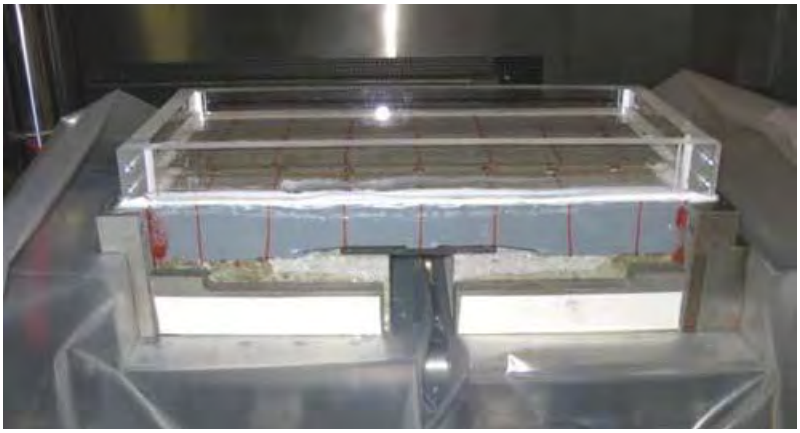


Movement testing at -40° to +60°C

The specimen was also subjected to 7,500,000 sinusoidal cycles, with an amplitude of 1 mm, at ambient temperature and a frequency of 5 Hz. In addition, dynamic properties were voluntarily tested at -40°C. The dynamic behaviour of the material was shown to be excellent, with the specimen showing no irregularities or signs of fatigue after the testing.

Watertightness testing

After successfully passing the aforementioned movement testing, the full-scale specimen was subjected to a water tightness test at the BAM institute in Berlin. At the maximum opening position of the joint, water was applied to a level of 30 mm above the highest point of the joint and maintained at that level for six hours. After the test, no signs of leakage or moisture could be found under the specimen.

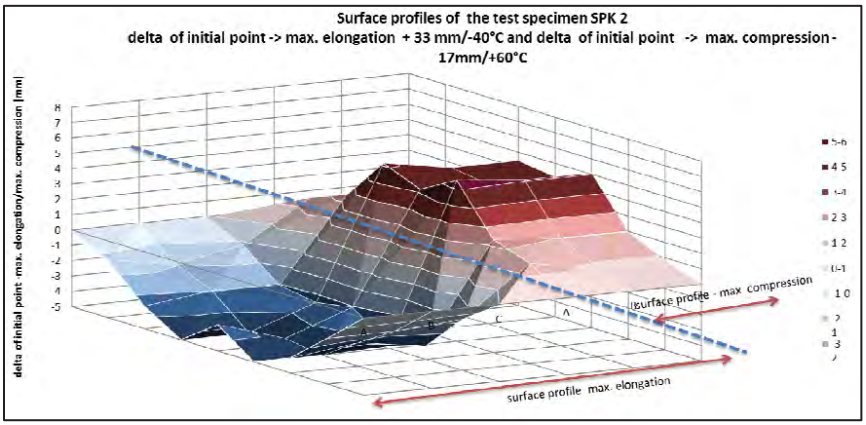


Verification of watertightness (at maximum opening position of joint) following movement capacity test, at the BAM institute, Berlin

Measurement of level differences in the surface

The flatness of the full-scale specimen was checked prior to the above-mentioned tests, to verify that any deviations in the level of the driving surface from the ideal connection line between the two adjacent pavements (without any imposed horizontal deformations and in the unloaded condition) are not greater than 5 mm – in accordance with the Austrian standard RVS and the relevant ETAG.

After loading, greater deviations are permitted, but these must not exceed 10 mm. Verification checks were carried out during and after both the fatigue and movement tests as described above. The results were positive, with a maximum level increase of +6 mm and a maximum level decrease of -5 mm being recorded.



Measurement of deviation from ideal connection line across joint in maximum opened and maximum closed position, at the BAM institute, Berlin

Skid resistance testing

The full-scale specimen was subjected to skid resistance testing with a portable skid resistance pendulum tester as described in EN 13036-4, using the CEN rubber slider for carriageways and the 4S rubber slider for footpath areas.

Further testing

Testing was carried out on the joint's components to establish durability characteristics as follows:

- Resistance to chemicals, such as oil, fuel and de-icing agents, according to EN ISO 175
- Temperature-based ageing: Various tests according to EN 13687-2, EN 13687-3, EN 13687-5
- Ageing resulting from UV-radiation and weathering: Long-term tests (3030 hours) to TR010
- Ageing resulting from ozone: Test according to ISO 1431
- Freeze-thaw test (with thaw salt) according to EN 13687 Part 1

Resulting European Technical Approval

As a result of this testing, the expansion joint has been awarded a European Technical Approval (ETA). This ETA covers joints of this type that accommodate longitudinal movements (SLS) of up to 135 mm (90 mm expansion and 45 mm compression), with a thickness of 60 mm and an initial width of 1100 mm. All types are designed for a vertical displacement of +/- 10 mm, permitting bridge bearing replacement work to be carried out without damaging the joint.

Member of EOTA

European technical approval ETA-12/0260

(English translation, the original version is in German)

Handelsbezeichnung *Trade name*	POLYFLEX® Advanced PU
Zulassungsinhaber *Holder of approval*	Reisner & Wolff Engineering GmbH Terminalstraße 25 4600 Wels Austria
Zulassungsgegenstand und Verwendungszweck	Elastische Belagsdehnfuge für einen nominellen Dehnweg von 15 mm – 135 mm
Generic type and use of construction product	*Flexible plug expansion joint for nominal movement capacity of 15 mm – 135 mm*
Geltungsdauer vom Validity from bis zum to	26.07.2012 25.07.2017
Herstellwerk *Manufacturing plant*	Auflistung der Herstellwerke festgelegt in der technischen Dokumentation *Comprehensive list of manufacturing plants laid down in technical documentation*

Further national approvals have also been awarded. For example, an approval issued by NEXCO (the West Japan Highway Administration) on the basis of specially completed testing have validity right across Japan.

INDEPENDENT EVALUATION BY ENGINEERING FIRM ON BEHALF OF A CLIENT

In 2015, in the early stages of a large project in Switzerland (the Postparc development project in Berne's city center), a number of *Polyflex® Advanced* expansion joints were initially installed in order to

Core showing solid bonding to structure concrete

provide an opportunity for the client to evaluate their performance under the site's real service conditions. An independent engineering consultant investigated and assessed the performance of the joints, in particular with respect to watertightness, movements, rutting, penetration of point loads and adhesion to subsurface. Images from some of the tests carried out are shown in the photographs.

Testing of penetration of a point load (long bar at center). On right, deformation has fully reversed.

The engineers' report concludes with the following recommendation:

The engineering partnership recommends the use of Polyflex®Advanced for the expansion joints, for the following reasons:

Engineering partnership SCHANZENPOST

Henauer Gugler AG, Berne (lead)
Hartenbach & Wenger AG, Berne

PostParc Berne
Berne railway station development project

Expansion joints
Evaluation of pilot project with *Polyflex Advanced* system

Hartenbach & Wenger AG
Egelgasse 70
3006 Bern

Berne, 26.08.2014

P. Schmied / P. Furrer

- The system has been widely used, especially in Austria, and can present the most references to date. No problems were experienced during the installation of the test strips.
- The most important requirements were satisfied by the test strips (watertightness, bonding to different materials, movement capability).
- The system is practically the only one which can be recommended today, even if a certain degree of maintenance can be expected during its service life. Expansion joints are mechanical parts that are subjected to heavy wear and tear. The systems of other suppliers may well offer the same properties in the near future, but are not yet ready to be recommended for such a large application.

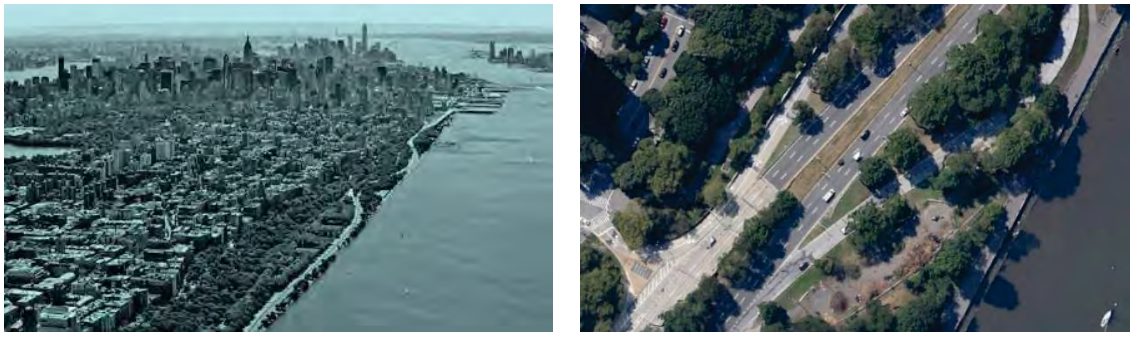
APPLICATIONS

In general, *Polyflex®Advanced* expansion joints may be used for almost any small-movement application (new-build or refurbishment), but the following cases are worthy of special mention:

- Railway bridges as well as road bridges
- Architectural applications
- Pedestrian areas, cycle lanes
- Areas where cleanliness is very important, e.g. pharmaceutical and food processing industries, hospitals and laboratories
- Areas where resistance to acids and bases is required, e.g. chemical industry
- Replacement of existing expansion joints

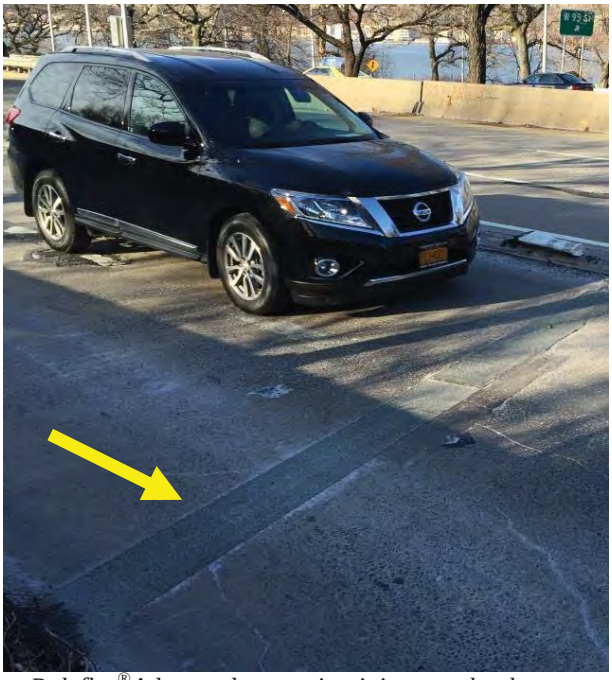
REFERENCE PROJECT: HENRY HUDSON PARKWAY, NEW YORK – 95TH STREET OFF-RAMP

A project was implemented for New York City Department of Transport (NYCDOT) at the end of 2014, with the replacement of an old small-movement expansion joint at the 95th Street off-ramp (southbound) of the Henry Hudson Parkway in Manhattan. The joint was installed in two phases: One lane in November 2014 and the second a month later.



Location of the project on Manhattan's Upper West Side

Existing expansion joint after removal of one particularly deteriorated section



New Polyflex® Advanced expansion joint seamlessly "disappearing" in the road surface

The same program was followed for both lanes. One the first day, the traffic lane was closed in the morning and the old joint was removed, and then the surfaces were sandblasted and the steel angles and anchors were installed. The next day, the *Polyflex® Advanced* material was poured and levelled, and the lane was opened to traffic in the afternoon. Considering the cold temperatures due to the time of year and the resulting longer curing time required, steel plates were placed across the joint to protect the still-curing material during the first hours of service. Subsequent inspections have shown that the joint has performed well under traffic, with no signs of wear and tear or ageing.

CONCLUSIONS

The *Polyflex® Advanced* expansion joint offers all the benefits of the traditional asphaltic plug expansion joint – including smooth, safe, low-noise surface, great adaptability and easy installation. However, it overcomes the numerous disadvantages and challenges that have always plagued asphaltic plug joints. It offers enormously improved strength, elasticity and durability, resulting in much less maintenance and far more reliable watertightness. Installation is far easier and less prone to error, with the two-component compound being mixed at ambient temperatures. And damage to previously placed material can be easily and reliably repaired by pouring of new PU material onto the old, damaged material, chemically reactivating it. The speed of installation and curing of the material is particularly beneficial when the joint

is installed on an existing structure to replace an old expansion joint, as the amount of demolition required is minimal and the construction and material curing time is so short, minimizing disruption to traffic. For these reasons and more, this modern plug-type expansion joint is likely to be increasingly used in years to come, in both bridge construction and bridge maintenance.

G-18: ADVANCED MATERIALS FOR ABC

APPLICATIONS AND USE OF CSA CEMENT IN ACCELERATED BRIDGE CONSTRUCTION

Ken Vallens, Chris Davis, CTS Cement Manufacturing Corp., Headquarters: 11065 Knott Avenue, Cypress, CA 90630, 800-929-3030, cdavis@ctscement.com

INTRODUCTION

With continued exponential growth of the world's population affecting a significant increase in demand for infrastructure of water, roads, buildings, housing and utilities to support economic growth and quality of life (1), combined with aging infrastructure that has reached arresting levels of disrepair and risk in the United States and around the world (2), challenges mount for the built environment to ensure these critical needs are met while exercising good stewardship of natural resources and of the investment dollars of both the private and public sectors. Planning, design, construction, facility management, and owner communities are tasked with creating a safer, more sustainable, environmentally conscious, and economical built environment. This undertaking requires "materials and systems [that] foster change in sustainable development practices in the materials, design and construction industry through the use of high-performance materials and systems that improve function, durability, and safety while minimizing life-cycle costs and environmental impact."(3)

Maximizing asset lifecycles and design/build efficiencies while minimizing environmental impacts, operation and maintenance costs, and initial project costs are paramount. To achieve these formidable objectives, projects must be engineered to provide extended service life by designing for durability, and preventing deterioration and failure.

As the world's most widely used construction material (4), concrete has come under pivotal scrutiny to ensure high quality, durable, low maintenance, sustainable concrete designs. Calcium sulfoaluminate (CSA) cement technology provides performance advantages that offer significant contributions to increased durability and extended service life of concrete assets by preventing deterioration caused by shrinkage and chemical sulfate attack, while maximizing durability and service life. Its ease of use and compatibility with common industry materials and other complementary products ensure today's engineered concrete designs will substantially contribute to tomorrow's quality of life and economic opportunity.

HISTORY

Hydraulic calcium sulfoaluminate (CSA) cement is a modified derivative of portland cement clinker developed in the 1950s by Professor Alexander Klein at the University of California – Berkeley, with collaborative development efforts furthered with the University's Edward H. Rubin and Dr. Edward K. Rice. Early initial research was inspired by the need for a higher quality cement that would overcome the shortfalls of portland cement – namely excessive shrinkage, susceptibility to chemical attack, destructive reactions with certain aggregates, and negative consequences of traditional accelerating admixtures. Applications were focused primarily on the shrinkage compensation of portland cement concrete with what is now known as Type K Shrinkage-Compensating Concrete, and chemically pre-stressed concrete applications.

In the 1960s, Professor Klein and Dr. Rice further developed CSA technology and invented a rapid hardening hydraulic cement (ASTM C1600)(5) that offered both early strength gain and long-term strength retention without the addition of portland cement. Combined with other key performance advantages, CSA cement now offered the concrete industry a remarkably more durable and higher performing cement solution.

The first industrial production of CSA cement was introduced in the U.S. market in the 1960's and was embraced by cement and concrete producers seeking to maximize production opportunity and capitalize on the post- World War II economic expansion. Over the next 20 years, CSA cement enjoyed notable exposure in many high-profile projects that incorporated the technology as either Type K Shrinkage-Compensating Concrete or Rapid Hardening Hydraulic Cement Concrete and other concrete repair materials. Notable projects include: the Los Angeles World Trade Center, the District of Columbia Subway System, the Chicago O'Hare Parking Structure, Dallas City Hall Building, and the Ohio Turnpike Bridge Decks. After the 1994 Northridge, CA Earthquake, Rapid Hardening Hydraulic Cement Concrete and concrete repair materials were used to repair mass structures and damaged interstate sections that allowed these essential infrastructure elements to be expeditiously returned to service.

On-going investment in research and development of CSA technology continues today, with bulk, contractor-based, and consumer-based concrete and concrete repair products available throughout the United States and around the world.

TECHNOLOGY

Calcium sulfoaluminate cement is a pure hydraulic cement, manufactured with similar raw materials (i.e., limestone, clay and gypsum), equipment, and processes that are used to make Portland cement with some

important distinctions. The manufacture of CSA cement requires more extensive refinement and quality control, and precise selection and proportioning of high quality raw materials. Once selected and proportioned, the raw materials are burned at lower kiln temperatures than portland cement, with these key elements combining to produce CSA cement's unique chemistry and superior performance properties.

Belitic CSA cement is primarily composed of hydraulic tetracalcium trialuminosulfate (CS) (commonly referred to as calcium sulfoaluminate, or CSA) and dicalcium silicate (CS) (often referred to as belite). The ($C_4A_3\bar{S}$) compound hydrates quickly, maximizing ettringite formation and early strength gain during the first seven (7) days. CSA forms only needle-like ettringite crystals that create a tight network of structural crystals that maximizes strength and durability. This rapid formation of ettringite crystals and rapid strength gain is what provides the fast performance and quick in-service turnaround of Rapid Hardening Hydraulic Cement materials.

In portland cement concrete, C_3A is the compound most commonly attacked by sulfates, leading to degradation and failure of the concrete. In addition to the use of tetracalcium trialuminate sulfate ($C_4A_3\bar{S}$) to achieve rapid strength gain in the early stages of placement, and dicalcium silicate (C_2S) that contributes to long-term strength, CSA offers an important advantage – the absence of tricalcium aluminate (C_3A), making it extremely resistant to sulfate attack.

As noted in ACI 223(6), "Because it is the C_3A that is attacked by sulfates, the concrete vulnerability can be reduced by using cements low in C_3A."

Independent testing conducted by CTLGroup(7), an independent testing laboratory that serves the scientific, engineering, consulting, and design industries, found the C_3A content of CSA cement to be undetectable. This absence of C_3A is the key factor in its superior performance in sulfate resistance when compared to other portland cements.

Another important distinction of CSA cement is in its use of mix water. Traditional portland cement concrete and CSA cement concrete both require similar amounts of mix water (typically 0.45 w/c) to ensure workability during placement and finishing. However, portland cement concrete (PCC) only uses approximately 55% of that water (0.25 w/c) in the formation of its calcium silicate hydrate (CSH) hydration product. The remaining "water of convenience" creates a medium where heavier particles can settle and segregate, displacing the unused "bleed" water. This excess water creates voids and capillary channels during egress to the surface which, in turn, creates shrinkage and points of entry for contamination after the concrete has hardened. Left unprotected, contaminants penetrating the concrete

through this network of bleed water channels (capillaries) leads to degradation of steel reinforcement, and ultimately, to concrete failure.

In contrast, the hydration mechanism in CSA consumes 98+% of the water molecules and chemically binds them within the ettringite structure. Portland cement's hydration mechanism merely adheres them via molecular adsorption (i.e., adhesion to the exterior of the ettringite crystals and calcium hydroxide platelets). By completely binding the water molecules within the ettringite structures, CSA more efficiently consumes and utilizes water molecules in the mix, and maximizes ettringite formation and concrete strength. This, in conjunction with CSA's slightly expansive nature, effectively eliminates excess water and their associated voids and capillary channels that result in drying shrinkage, contamination and deterioration. Combined with the slightly expansive nature of the CSA compound, CSA helps overcome the shrinkage and edge curling common in Portland cement concrete resulting from water on the surface that evaporates quicker than at the base of the slab.(8)

SUMMARY

CSA cement concrete offers substantial performance advantages versus portland cement concrete.

Its versatility makes it suitable for use in a wide range of industries and applications – whether new construction or repair and rehabilitation. It is used in a wide range of concrete and concrete repair materials and is readily available in both the U.S. and global markets.(9)

Calcium sulfoaluminate cement technology is most commonly used in two primary forms: (1) In fast-setting hydraulic cement materials (ASTM C1600 - Standard Specification for Rapid Hardening Hydraulic Cement) and (2) in Type K Shrinkage-Compensating Concrete (ACI 223). The CSA compound remains the same, though its use and function are tailored to create a wide variety of CSA material solutions.

When engineered for use in fast-setting hydraulic cement concrete, CSA provides a fast, initial set within 20 minutes, and the ability to achieve structural strength in one hour. This fast turnaround minimizes costly downtime and speeds time to service, affecting a quick return on investment and profitable asset use. The performance characteristics of CSA cement technology offer an array of material options for any concrete project. The design, construction, owner, developer, and manufacturing communities can meet long-term industry initiatives to maximize asset lifecycles and minimize environmental impacts using this proven technology.

This presentation will focus on the practical application of CSA cement in Accelerated Bridge Construction projects.

ACKNOWLEDGEMENTS

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Cement Materials Science & Engineering Team: Grant Kao, John Kim, Nick De Ocampo, Danny

Chung. Any opinions, findings and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of other professional participants or institutions.

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RECENT CHANGES IN AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS REGARDING LIGHTWEIGHT CONCRETE

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ABSTRACT

Lightweight concrete has obvious benefits in reducing the weight of precast concrete elements often used for ABC projects. However, some engineers have found that current design provisions make the use of the material more costly and less efficient. Revisions to the *AASHTO LRFD Bridge Design Specifications*, approved in 2014 and 2015, have simplified design using lightweight concrete and now allow its potential to be better realized in bridge designs. This paper presents these changes related to lightweight concrete along with background data used to justify the changes. The effect of the changes is illustrated with several numerical examples.

INTRODUCTION

The American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS) Technical Committee 10 (T-10) revised articles in the *AASHTO LRFD Bridge Design Specifications* relating to lightweight concrete (LWC) in their April 2015 meeting. These revisions were the result of a research program conducted by the Federal Highway Administration (FHWA) at the Turner-Fairbank Highway Research Center (TFHRC) to investigate the performance of modern lightweight concretes. This effort engaged the academic, public sector, and private sector communities to compile the body of knowledge on lightweight concrete while also conducting nearly 100 full-scale structural tests on lightweight concretes. (1-3)

This paper describes the recent revisions made to the *AASHTO LRFD Bridge Design Specifications* (4) regarding LWC with a focus on why the changes were made and how the changes will affect the design process. The revisions included a change in the definition of LWC, a change to the expression for concrete modulus of elasticity, the addition of a concrete density modification factor, and a change to the resistance factor for the shear resistance of LWC members.

LIGHWEIGHT CONCRETE DEFINITION

Traditional LWC either had lightweight coarse and lightweight fine aggregate, termed “all-lightweight concrete”, or lightweight coarse aggregate combined with normal weight fine aggregate, termed “sand-lightweight”. Unit weights ranging from 0.100 kcf to 0.120 kcf were common for these traditional LWC mixes. These traditional constituent materials and unit weights were included in the previous definition of LWC in the *AASHTO LRFD Bridge Design Specifications*.

Current use of reduced unit weight concrete also includes “specified density concrete” and “inverted mixes”. A specified density concrete has a blend of lightweight coarse and normal weight coarse aggregate and commonly uses normal weight fine aggregate. Specified density mixes commonly have a unit weight ranging from 0.120 to 0.135 kcf, which is between that of traditional LWC and normal weightconcrete (NWC), although they could be anywere in the range from all lightweight concrete to NWC. Inverted mixes use normal weight coarse aggregate and lightweight fine aggregate to achieve the reduced unit weight. LWC mixes like specified density concrete and inverted mixes were not addressed by the previous definitions for LWC in the *AASHTO LRFD Bridge Design Specifications*. This is because the previous LWC definitions were for sand-lightweight and all-lightweight concrete and limited the unit weight to a maximum of 0.120 kcf (1920 kg/m^3). Normal weight concrete was defined as having a unit weight from 0.135 to 0.155 kcf. With these definitions of LWC and NWC, there was a range of unit weights (0.120 to 0.135 kcf) where the applicable concrete type was not defined by the specifications. This left designers uncertain as to how to proceed when they wanted to use concrete unit weights in this range. The new definition for LWC is as follows:

“*Lightweight Concrete* – Concrete containing lightweight aggregate conforming to AASHTO M 195 and having an equilibrium density not exceeding 0.135 kcf, as determined by ASTM C567.” The new definition for LWC has expanded the range of unit weight up to the lower limit for NWC and eliminates the definitions related to the constituent materials used to make LWC. This means that any concrete with a unit weight below 0.135 kcf is defined as LWC and any concrete with a unit weight above 0.135 kcf is defined as normal weight concrete. This revision eliminates the gap in unit weight and gives the designer the freedom to specify any reasonable unit weight (generally greater than 0.095 kcf), including those slightly lower than NWC. In most cases, greatest economy is achieved by using concrete with unit weights in the range of 0.110 to 0.125 kcf.

CONCRETE MODULUS OF ELASTICITY

The previous expression for concrete modulus of elasticity (E_c) in the *AASHTO LRFD Bridge Design*

Specifications was empirical and fit the more limited data available when it was proposed by Pauw (5) in

1960. As part of the FHWA investigation at the TFHRC (1), a database of over 2500 measurements of elastic modulus on LWC available in the literature were collected. The database collected as part of NCHRP Project 12-64 (6) included 3800 elastic modulus measurements on NWC. The measurements on LWC specimens in the TFHRC database and the measurements on the NWC specimens in the NCHRP database were used to evaluate the previous AASHTO LRFD expression and the new expression adopted by the SCOBS T-10 committee. The new expression for E_c is given by Eq. 1. The notations for all expressions are given at the end of the paper.

$$E_c = 120{,}000K_1(w_c)^{2.0}(f'_c)^{0.33} \qquad \text{(Eq. 1)}$$

There is a considerable amount of scatter in measurements of concrete elastic modulus as can be seen in Figure 1 and Figure 2. Figure 1 compares the measured E_c values to predicted E_c values computed using the previous expression. Predicted values of E_c given by the new expression are compared to the measured E_c data in Figure 2. The coefficient of determination (R^2) (7) is shown in both figures. The coefficient of determination is a measure of the quality of the prediction expression and a larger R value indicates a better fit to the measured data. The previous expression overestimated 67% of the E_c measurements on LWC specimens and nearly 19% of the measurements on LWC specimens were overestimated by more than 20%. The new expression overestimates 42% of the E_c measurements for LWC specimens, and only 9% of the measurements were overestimated by more than 20%.

Both expressions give similar predictions of E_c for NWC with a compressive strength up to 10 ksi. The revisions to the commentary allow the previous expression for E_c to be used for NWC with specified compressive strengths up to 10 ksi. Nearly all of the NWC data with compressive strengths greater than 15 ksi were over-estimated by the previous expression.

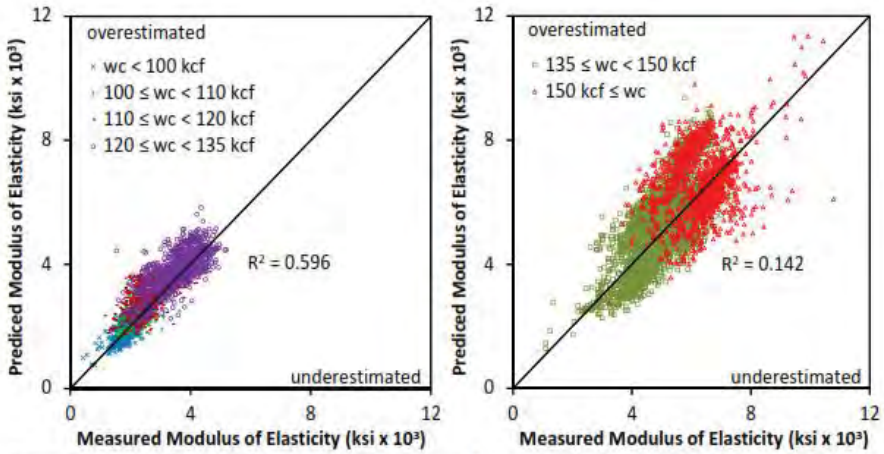

Figure 1. Comparison of Measured and Predicted Concrete Elastic Modulus Determined using Previous AASHTO LRFD Expression.

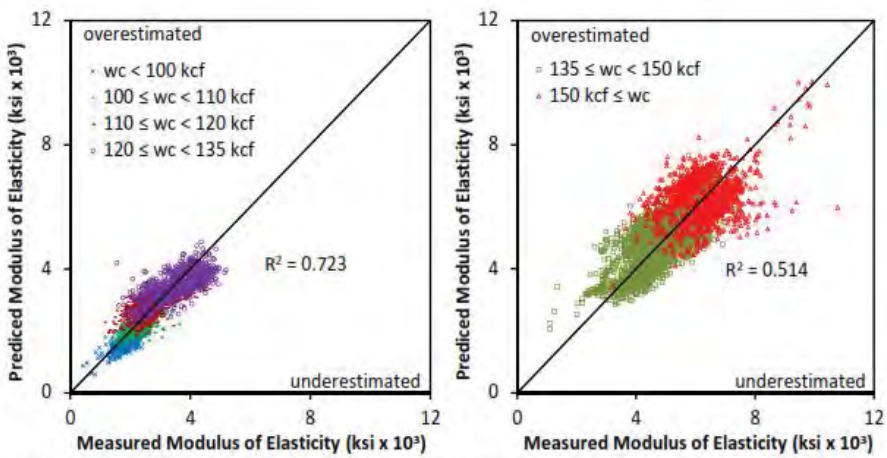

Figure 2. Comparison of Measured and Predicted Concrete Elastic Modulus Determined using New AASHTO LRFD Expression.

CONCRETE DENSITY MODIFICATION FACTOR

The potential reduction in tensile strength of LWC was not consistently treated in the *AASHTO LRFD Bridge Design Specifications*. The expressions in Articles 5.8.2 and 5.8.3 pertaining to shear resistance used factors of 0.75 and 0.85 for all LWC and sand LWC, respectively. The development length of mild steel in tension in Article 5.11.2.1.2 included a factor for lightweight concrete that increased the development length.

Other articles had separate expressions for LWC and NWC, such as Article 5.4.2.6 for modulus of

Some articles that dealt with the tensile strength of concrete did not specify any different requirements for

LWC and NWC, such as Article 5.9.4 for stress limits of prestressed concrete.

A single factor was defined in the revisions to the *AASHTO LRFD Bridge Design Specifications* to account for the potential reduction in tensile strength of LWC. The concrete density modification factor, or λ-factorwas defined based on either the specified unit weight or specified splitting tensile strength. The λ-factor is now included in any appropriate article and equation where concrete tensile strength contributes to nominal resistance. A designer using LWC can now specify concrete compressive strength and unit weight without regard to the types of constituents used to achieve the unit weight reduction. The specified unit weight is typically used to determine the λ-factor, which is then used throughout Chapter 5 of the specifications. The expression for the λ-factor based on unit weight is given by Eq. 2. If the concrete splitting tensile strength (f_{ct}) is specified, it can be used to determine the λ-factor using Eq. 3. The expression given by Eq. 2 is illustrated schematically in Figure 3.

$$0.75 \leq \lambda = 7.5w_c \leq 1.0 \qquad \text{(Eq. 2)}$$

$$\lambda = 4.7f_{ct} \leq 1.0 \qquad \text{(Eq. 3)}$$

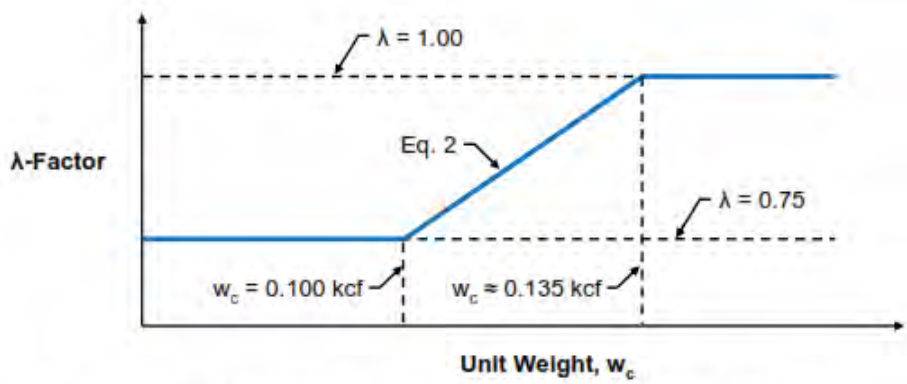

Figure 3. Illustration of λ-Factor as a Function of Unit Weight.

The ratio of f_{ct} to the square root of the compressive strength is known as the splitting ratio, F_{sp}. The term "splitting ratio" is no longer used in the *AASHTO LRFD Bridge Design Specifications*, but the definition was previously part of the modification factor for LWC in Articles 5.8.2.2 and 5.11.2.1.2 (4). Concrete with a F_{sp} greater than 0.212 did not require modification of the expressions in Articles 5.8.2 and 5.8.3 for LWC. F_{sp} implied by the *AASHTO LRFD Bridge Design Specifications* for sand-lightweight concrete and all-lightweight concrete were based on the 0.85 and 0.75 modification factors described previously in Article 5.8.2.

The splitting ratios for over 1300 specimens are shown in Figure 4. The splitting ratios for the all lightweight concrete specimens are shown in the figure. A LWC modification factor of 0.75 is represented by the horizontal line at a splitting ratio of 0.16. A horizontal line at a splitting ratio of 0.18 represents the LWC modification factor of 0.85 for sand-lightweight concrete. Two vertical lines at unit

weights of 0.120 and 0.135 kcf represent the limits of the gap of unit weights not specifically addressed in the previous edition of the *AASHTO LRFD Bridge Design Specifications* (4). A solid line represents the splitting ratios implied by Eq. 2 for the λ-factor.

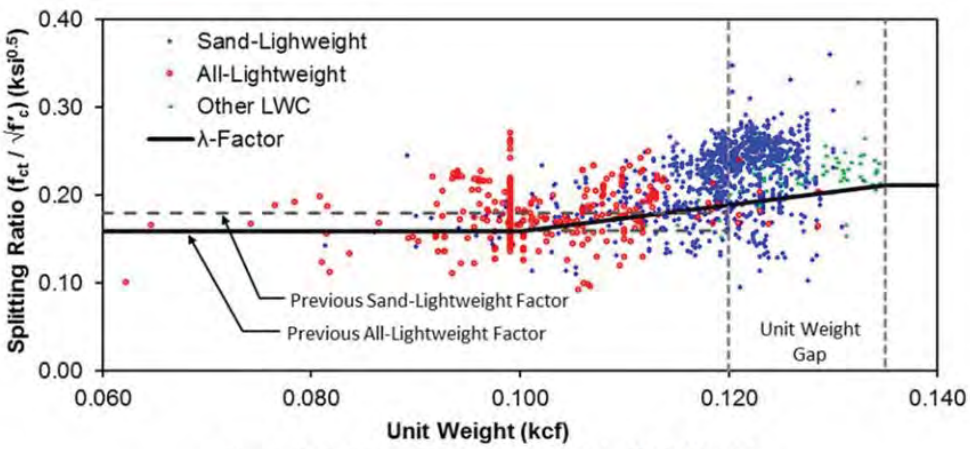

Figure 4. Splitting Ratio Compared to Unit Weight.

APPLICATION OF THE CONCRETE DENSITY MODIFICATION FACTOR

The λ-factor was inserted into many of the expressions in Chapter 5. This section will discuss the expressions for concrete modulus of rupture, nominal shear resistance, and development length of mild steel in tension that were evaluated as part of the FHWA investigation.

The λ-factor was also added to many other expressions in Chapter 5, but these expressions are not discussed in this paper. These expressions include the cracking torque in Article 5.8.2.1, the stress limits of prestressed concrete in Article 5.9.4, the shear resistance of piers in Article 5.10.11.4, the shear resistance of brackets, corbels, and beam ledges in Article 5.13.2, the shear resistance of box culverts in Article 5.14.5.3, and the tensile and shear strength of segmental box girders.

Concrete Modulus of Rupture

Before the April 2015 revisions, the *AASHTO LRFD Bridge Design Specifications* had different expressions for the modulus of rupture (f_r) depending upon the purpose of the calculation and the type of concrete. For NWC, there was on expression for f_r to be used when computing nominal shear resistance and another expression for f_r to be used in expressions for effective moment of inertia, cracking control, and minimum flexural reinforcement. For LWC, there were separate expressions for sand-lightweight concrete and all-lightweight concrete. Unlike for NWC, the *AASHTO LRFD Bridge Design Specifications* did not give different expressions for the modulus of rupture of LWC depending on the purpose of the calculation. This created varying levels of conservatism in the calculations of crack control, effective

moment of inertia, and cracking moment for shear resistance when used in members made from LWC. The new expressions for modulus of rupture are given by Eq. 4 for use in all articles except those for nominal shear resistance when Eq. 5 should be used. These two expressions are essentially the same as the previous expressions for NWC with the addition of the λ-factor.

$$f_r = 0.24\lambda\sqrt{f'_c} \qquad \text{(Eq. 4)}$$

$$f_r = 0.20\lambda\sqrt{f'_c} \qquad \text{(Eq. 5)}$$

Nominal Shear Resistance

The previous expressions relating to nominal shear resistance were intended to be modified for use with LWC. Article 5.8.2.2 stated that $\sqrt{f'_c}$ in Articles 5.8.2 and 5.8.3 should be replaced with $0.85\sqrt{f'_c}$ or $0.75\sqrt{f'_c}$ for sand-lightweight and all-lightweight concrete, respectively. However, it was not always clear to designers if this was intended to apply only to expressions used to compute the nominal shear resistance (V_n), or also to other expressions in Articles 5.8.2 and 5.8.3 such as those to calculate minimum shear reinforcement in Article 5.8.2.5 or to calculate cot θ in Article 5.8.3.3. The revised *AASHTO LRFD Bridge Design Specifications* include the λ-factor in all appropriate articles related to nominal shear resistance. There are currently three basic procedures in the *AASHTO LRFD Bridge Design Specifications* to calculate the nominal shear resistance provided by tensile stress in the concrete (V_c). These are the Simplified Procedure for Nonprestressed Sections (Article 5.8.3.4.1), the General Procedure (5.8.3.4.2), and the Simplified Procedure for Prestressed and Nonprestressed Sections (5.8.3.4.3). If the designer chooses to use the Simplified Procedure for Nonprestressed Sections or the General Procedure, then V_c is determined using Eq. 6. If the designer choses to use the Simplified Procedure for Prestressed and Nonprestressed Sections, V_c is the lesser of V_{ci} and V_{cw} from Eq. 7 and Eq. 8. The λ-factor is included in all three of these expressions.

$$V_c = 0.0316\beta\,\lambda\sqrt{f'_c}b_v d_v \qquad \text{(Eq. 6)}$$

$$V_{ci} = 0.02\,\lambda\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\,\lambda\sqrt{f'_c}b_v d_v \qquad \text{(Eq. 7)}$$

$$V_{cw} = \left(0.06\,\lambda\sqrt{f'_c} + 0.30 f_{pc}\right)b_v d_v + V_p \qquad \text{(Eq. 8)}$$

The λ-factor was intentionally not included in the expression for cot θ in Article 5.8.3.4.3 as given by Eq.9.

$$\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8 \qquad \text{(Eq. 9)}$$

This expression is included in Article 5.8.3 and therefore, according to the 7th Edition of the *AASHTO LRFD Bridge Design Specifications* (4), the $\sqrt{f'_c}$ should be replaced by $0.85\sqrt{f'_c}$ or $0.75vf'_c$. If the λ-factor had been inserted in front of the $\sqrt{f'_c}$ in Eq. 9, then the nominal shear resistance (V_n) determined for a LWC prestressed girder using the Simplified Procedure for Prestressed and Nonprestressed Sections could be higher than the shear resistance determined for an identical NWC girder. An identical girder would have the same cross section, design material properties, reinforcement, and internal forces.

Development Length of Mild Reinforcement

The articles pertaining to the development length of mild reinforcement underwent substantial changes in the 2015 Interim Revisions to the *AASHTO LRFD Bridge Design Specifications* (4). The expression for basic development length (l_{db}) was revised, but the 1.3 multiplier for LWC remained the same. The additional revisions from the April 2015 SCOBS T-10 meeting included the new λ-factor. The new expression for development length (l_d) is given by Eq. 10. The λ-factor is included in the denominator so that LWC with a λ-factor less than 1.0 will increase the development length.

$$\ell_d = \ell_{db} \times \left(\frac{\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc} \times \lambda_{er}}{\lambda}\right) \qquad \text{(Eq. 10)}$$

RESISTANCE FACTOR FOR LWC IN SHEAR

The first edition of the *AASHTO LRFD Bridge Design Specifications* had resistance factors for shear of 0.9 for NWC members and 0.7 for LWC members. By the sixth edition, the resistance factor for LWC had been increased to 0.8 based on an evaluation of a limited number of LWC and NWC beams in shear (8). The FHWA investigation included a reliability analysis of LWC and NWC members (2). Both reinforced concrete and prestressed members were evaluated. The analysis evaluated the uncertainty in the applied loads, the uncertainty in the fabricated cross section dimensions, and the uncertainty in the expressions used to predict shear resistance in the *AASHTO LRFD Bridge Design Specifications*. The analysis showed that a resistance factor of 0.90 can be used for both LWC and NWC with similar levels of reliability.

EFFECT OF NEW LWC PROVISIONS ON BRIDGE DESIGNS

Most of the recent changes to the *AASHTO LRFD Bridge Design Specifications* related to LWC will not result in significant changes to LWC bridge designs. The effect of the changes on most of the quantities discussed above are considered in this section.

The implementation of a new definition for LWC will not have a noticeable effect on bridge designs.

However, it will provide designers with greater freedom in specifying concrete densities without consideration of the type of materials that will be used by the concrete suppliers. This should make designers more comfortable with using LWC mixtures, which may result in the use of LWC for some structures where it would not have been considered in the past.

The effect of the new modulus of elasticity equation is illustrated in Figure 5 where the computed modulus is plotted versus concrete compressive strengths for NWC (0.145 kcf) and for all LWC (0.100 kcf).

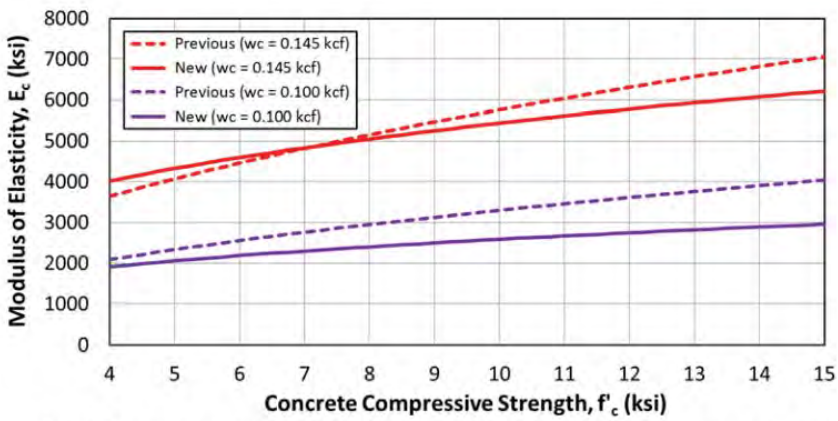

Figure 5. Comparison of Modulus of Elasticity computed using Previous and New Equations.

This figure shows that the modulus of elasticity is reduced using the new equation for higher strengths of NWC, but the modulus is reduced for all strength levels for all LWC. For prestressed concrete members, this will lead to the calculation of slightly larger prestress losses and cambers.

The change in the definition of the density modification factor will not have a significant effect on bridge designs using LWC because the modification factors have been present in previous editions of the AASHTO Standard and LRFD specifications. However, having a definition of the modification factor that is based on density will be helpful, eliminating the confusion that accompanied the use of the constituent-based definitions of concrete type to determine the modification factor in previous specifications. It is also very helpful that the density modification factor has been assigned a variable name, λ, an assigned variable, the factor can be inserted in all equations and expressions where it is appropriate. This simplifies and clarifies design for engineers, giving them more confidence that they can correctly design a bridge using LWC.

The effect of applying the concrete density modification factor, λ, in equations where it has not previously been included will not be large since the factor ranges from 0.75 to 1.00. In some cases, such

as the limiting tensile stress in the precompressed tensile zone of prestressed concrete elements, the stress limit is already fairly low and the reduction will not be significant in absolute terms. It was appropriate and necessary to uniformly insert the density modification factor into all equations and expressions where the assumed reduction in tensile strength that has historically been associated with the use of LWC could affect the computed quantity.

The most significant change that will result from the new revisions related to LWC is the elimination of the different shear resistance factor for LWC that has been present in the *AASHTO LRFD Bridge Design Specifications* since it was introduced in 1994. In the first edition, the shear resistance factor for LWC was 0.7 while the factor for NWC was 0.9. In 2011, the resistance factor for LWC was increased to 0.8. With the new changes, the shear resistance factors for NWC and LWC are both 0.9. Since the introduction of the

LRFD specifications, several designers have reported that the lower shear resistance factor for LWC had precluded the use of LWC because the benefits of the lower unit weight were counteracted by the reduced shear capacity. In one case, which was a concrete segmental box girder bridge, the reduced shear capacity for LWC required an increase in the web thickness which more than offset the weight reduction, making a LWC design uneconomical.

CONCLUDING REMARKS

Significant changes have been made in the *AASHTO LRFD Bridge Design Specifications* related to LWC.The reasons for the changes have been discussed, as well as some of the expected effects of the changes on designs using LWC. While the effects of the changes on bridge designs are not generally expected to be large, the changes provide a more rational and understandable basis for bridge design using LWC. This is expected to make the use of LWC more attractive to designers, hopefully allowing them to consider this material that may result in more economical bridge designs in the near future.

ACKNOWLEDGEMENTS

This document presents results from a research program and is intended to both facilitate broader understanding of the performance of lightweight concrete and to assist AASHTO SCOBS T-10 as they consider relevant revisions to Chapter 5 of the *AASHTO LRFD Bridge Design Specification* (4). It does not constitute a policy statement or a recommendation from FHWA. Additionally, the publication of this article does not necessarily indicate approval or endorsement of the findings, opinions, conclusions, or recommendations either inferred or specifically expressed herein by FHWA or the United States

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NOTATION

The following notation is used in the paper:

b_v = effective web width (inch)

d_v = effective shear depth (inch)

E_c = modulus of elasticity of concrete (ksi)

f'_c= concrete compressive strength in reference to material tests values and specified compressive strength in reference to articles of the AASHTO LRFD Specification (ksi)

f_{ct}= concrete splitting tensile strength (ksi)

f_{pc} = compressive stress at the centroid of the concrete after all prestress losses have occurred(ksi)

F_{sp} = concrete splitting ratio

f_r = modulus of rupture of concrete (ksi)

K_1 = correction factor for source of aggregate

l_d = development length (inch)

l_{db} = basic development length for straight reinforcement to which modification factors are applied to determine the development length (inch)

M_{cre} = moment causing flexural cracking at the section due to externally applied loads (kip-inch)

M_{max} = maximum factored moment at the section due to externally applied loads (kip-inch)

R^2 = coefficient of determination

V_c = nominal shear resistance provided by tensile stresses in the concrete (kip)

V_{ci} = nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment (kip)

V_{cw} = nominal shear resistance provided by concrete when inclined cracking results from

excessive principal tension in web (kip)

V_d = shear force at the section due to unfactored dead load (kip)

V_n = nominal shear resistance of the section (kip)

V_p = component of the effective prestressing force in the direction of the applied shear, positive if resisting the applied shear (kip)

w_c = concrete unit weight (ksi)

ß = factor indicating ability of diagonally cracked concrete to transmit tension and shear

θ = angle of inclination of diagonal compressive stresses (degrees)

λ = concrete density modification factor

λ_{rl}= reinforcement location factor

λ_{cf}= coating factor

λ_{rc}= reinforcement confinement factor

λ_{er}= excess reinforcement factor

ϕ = resistance factor

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LIGHTWEIGHT AGGREGATE AS GEOTECHNICAL FILL FOR ABC PROJECTS

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ABSTRACT

Construction of roadway approach fills can be on the critical path for accelerated bridge construction (ABC) projects if the fill will be placed on soft soils. In this situation, use of lightweight structural fill can accelerate construction by reducing both the amount of settlement and the time required for settlement to occur. This paper discusses the use of structural lightweight aggregate for geotechnical applications. Properties of lightweight aggregate related to geotechnical design and construction are presented. Two numerical examples are given. Several projects where lightweight aggregate has been successfully used for geotechnical applications in transportation projects are highlighted.

INTRODUCTION

Most bridge engineers think of the construction and erection of bridge components as the items on the critical path when trying to accelerate construction or repair of bridges. However, the construction of roadway approach fill can sometimes be on the critical path if the fill must be placed on soft soils. In this case, the use of lightweight fill material can accelerate construction by reducing both the amount of settlement and the time required for the settlement to occur. Lightweight aggregate can also be used as a quick and effective solution to reduce the load on an abutment that may be moving or cracking due to excessive soil pressure behind the wall or to reduce the load on an existing buried structure over which new fill is being placed.

Lightweight aggregate has been successfully used for 50 years as a lightweight structural fill material for a wide range of geotechnical applications including bridge projects. This paper summarizes the properties of structural lightweight aggregate that make it a durable material for geotechnical fill applications and briefly discusses the requirements for installation of lightweight aggregate fill, which are the same as required for installation of any crushed stone fill. Several examples of projects where lightweight aggregate has been used for bridge projects will be included, including the secured entrance ramp into the Pentagon that had to be constructed quickly using MSE wall elements after the 9/11 attacks.

PROPERTIES OF LIGHTWEIGHT AGGREGATE FOR GEOTECHNICAL FILL

Lightweight aggregate is a granular fill material that can be used for geotechnical applications in the same way as any other type of crushed stone fill is used. Lightweight aggregate is manufactured by expanding shale, clay or slate at high temperatures to form a hard, light structural aggregate. The material is available from suppliers across the US.

The engineering properties of lightweight aggregate related to its use as a geotechnical fill are (1):

- *Lower weight*: In-place compacted moist densities range between 40-65 lbs/ft^3 which is less than ½ that of ordinary fills

- *Improved stability*: The angle of internal friction (f) ranges from 35° to 45°+

- *Free draining*: Permeable; > 1 cm/sec assured because lightweight aggregate is a manufactured product

- *Abrasion resistant*: LA abrasion loss ranges from 20 - 40%

- *Soundness*: Typical magnesium sulfate soundness loss is less than 6%.

- *Durable*: The aggregate is tough and does not break down during handling.

- *Structural material*: Satisfies AASHTO M 195 and ASTM C330 specifications for structural

Lightweight aggregate Lightweight aggregate is installed in the same way as crushed stone fill.

- *Delivery*: Quick delivery using standard methods: truck, rail, barge

- *Simple*: Behaves like typical granular fill except easier to handle due to lighter weight

- *Standard equipment*: Loaders, static rollers and plate compactors are used to compact the aggregate

- *No formwork*: Just spread and compact

- *Not weather sensitive*: May be installed in any weather

- *Fast*: No wait for next lift

Lightweight aggregate can provide an economical and environmentally beneficial long-term solution to difficult geotechnical issues:

- *Twice the volume per truckload (by weight)*: Means 50% fewer trucks on the road or accessing a congested work site
- *Totally inert vitreous ceramic*: Will not degrade or change over time
- *No maintenance*: Long-term solution is achieved with one application
- *Infrastructure preservation*: May provide a rehabilitation option instead of mandatory replacement

NUMERICAL DESIGN EXAMPLES FOR EMBANKMENT FILL

The following is a numerical example that demonstrates the benefits of using lightweight aggregate for a 15-ft tall embankment fill. Three types of fill material are considered:

- Normal weight soil fill with an in-place unit weight of 110 pcf
- Normal weight select soil fill with an in-place unit weight of 120 pcf
- Lightweight aggregate fill with an in-place unit weight of 60 pcf

Using these three soils, the pressure applied on the soil beneath the embankment will be

- Normal weight soil fill 1650 psf
- Normal weight select soil fill 1800 psf
- Lightweight aggregate fill 900 psf

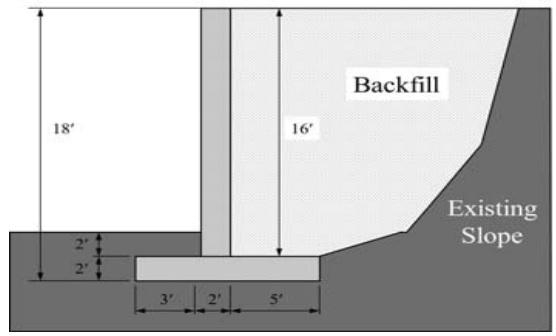
The lightweight aggregate fill reduces the weight of the fill by 45% and 50% for the normal weight soil fill and the normal weight select soil fill, respectively.

Another application of lightweight aggregate fill in embankment construction over poor soils is the use of the load balancing concept. If a 5-ft high embankment is to be constructed, using normal weight fill with an in-place unit weight of 110 pcf would produce an additional pressure on the existing soil of 550 psf.

This can result in undesirable settlement, which can lead to on-going maintenance issues. Assuming that the existing surface soil has a unit weight of 100 pcf, then 7.5 ft of existing soil can be removed and replaced with 12. 5 ft of lightweight aggregate. The 5-ft tall embankment is then achieved with no net change in pressure on the lower layers of soil. This means that there should be no settlement in the existing soil even though a 5-ft tall embankment has been constructed on it.

NUMERICAL DESIGN EXAMPLE FOR RETAINING WALL

The following is a numerical example that demonstrates the benefits of using lightweight aggregate as fill behind retaining wall. The reinforced concrete retaining wall shown in the figure to the right is designed to support 16 feet of backfill. The unit weight of concrete in the wall is taken as 150 pcf.



For the conventional soil design, the backfill is assumed to be a cohesionless sand with a total unit weight = 120 pcf and an angle of internal friction = 30 deg.

For the lightweight aggregate design, the backfill is a coarse grading of lightweight agg regate with a total unit weight = 60 pcf and an angle of internal friction = 42 deg.

Results of the design are shown in the following tables, which demonstrate a significant reduction in design parameters and a significant improvement in the factors of safety for the wall. This means that the wall design may be revised to provide a more economical solution, taking advantage of the reduced factors of safety.

	NW Sand	LWA	*LWA / NWA*
Active lateral earth pressure coefficient, K_a	0.33	0.198	*0.60*
The total active force per foot of wall, P_a	6.4 kips	1.92 k	*0.30*
The overturning moment per foot of wall, M_o	38.4 ft-k	11.52 ft-k	*0.30*
Resisting moment per foot of wall, M_r	107.3 ft-k	71.3 ft-k	*0.66*

Factor of safety against …	NW Sand	LWA	*LWA / NWA*
Overturning, FS_o	2.8	6.2	2.2
Sliding, FS_s	1.6	6.2	*3.9*
Bearing capacity failure, FS_b	2.6	6.3	2.4

EXAMPLES OF PROJECT WHERE LIGHTWEIGHT AGGREGATE HAS BEEN USED

The following is a sampling of projects where lightweight aggregate has been used in different bridge or transportation related applications. Several of the projects listed have contributed to accelerating the project completion through avoiding lengthy periods to allow settlement to occur or the use of soil remediation efforts that are also time consuming.

Pentagon Secured Entrance – Alexandria, VA

Following the attack on the Pentagon on 9/11, it was determined that a secured entrance was required for the facility. The construction schedule was very short. Details of the project are given in (2). During design, it was found that the bridge approach embankments crossed thick layers of compressible clay soils. If constructed u sing conventional materials (110 pcf), it was estimated that the fill would settle about 15 in. and that the settlement would take approximately 18 months. With a project schedule of only 22 months for the design/build project, this would not allow the team to complete the project on schedule. The geotechnical engineer proposed using lightweight aggregate for the fill in the MSE wall approaches.

Settlement for approaches using lightweight aggregate was estimated to be 6 in. with most of the settlement occurring within about 60 days. This allowed the project to be completed within the aggressive schedule.

Charter Oak Bridge – Hartford, CT

The Charter Oak Bridge links Hartford and East Hartford, Connecticut and was completed in 1991. The major structure carries U.S. Route 5 and State Route 15 over the Connecticut River. The soils in the project area were poor and large settlements were anticipated using conventional fill materials. Details of this project are reported in (3).

Lightweight fill was used to address four different situations encountered on the project:

- The approach fill for one abutment of the main bridge was over deep deposits of soft clay. Using lightweight aggregate reduced stresses on the poor soil, which reduced settlement. The higher shear strength of the lightweight aggregate also solved stability concerns. More than 80,000 cubic yards of lightweight aggregate were used in the 46-ft tall fill.

- Lightweight aggregate fill was used in approach embankments for another bridge to reduce settlement of an adjacent existing bridge.

- Soil over an existing 6.5-ft diameter culvert was removed and replaced with lightweight aggregate using the load balancing concept to offset the effects of the additional fill and pile driving and to avoid even a minor settlement of the aging structure.

- To improve the overall slope stability of an existing wharf consisting of an anchored sheet pile bulkhead, a 5-ft layer of existing soil was removed and replaced with lightweight aggregate fill.

Picardy Avenue Interchange – Baton Rouge, LA

The construction of the Picardy Avenue interchange with I-10 in Baton Rouge required tall retaining walls. However, the soils on the project site were poor. It was determined that normal weight fill could only be used for walls up to 22 ft tall, but the project required walls up to 40 ft tall. The project is discussed in (4).

Engineers determined that using lightweight aggregate for the fill behind the walls, which were constructed using segmental retaining wall block elements, would reduce the weight of the structures enough to avoid excessive settlement. Construction of the tall walls on the project required a total of over 120,000 cubic yards of lightweight aggregate.

NC 133 over Allen Creek – Wilmington, NC

NCDOT designed the repair of bridge approach fills on soft soil that were experiencing significant settlement. In some places, the asphalt thickness was 18 in., and the bridge had only been in service for 2 years (5). The existing soil was very soft. The repair project involved the removal of approximately 300 ft of roadway, embankment and existing soil on each approach which was replaced using lightweight aggregate. The reduced unit weight of the lightweight aggregate fill reduced settlement by making the weight of the fill and embankment approximately equal to the weight of existing soil removed, that is, by load balancing.

CONCLUDING REMARKS

Lightweight aggregate has been successfully used in geotechnical applications for nearly 50 years.

Delivery of many of bridge projects have been accelerated through its use by avoiding settlements or soil remediation efforts that would be caused by use of conventional fill materials.

This paper has introduced the material properties, demonstrated benefits of using lightweight aggregate by numerical examples, and has shown several bridge-related projects where lightweight aggregate was used to address geotechnical challenges.

ACKNOWLEDGEMENT

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USE OF LIGHTWEIGHT CONCRETE FOR BRIDGES MOVED INTO PLACE

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ABSTRACT

Construction of a bridge off-site and then moving it into final position by sliding or using self-propelled modular transporters (SPMTs) has become a more frequently used technique for ABC. Moving completed bridges presents several design challenges. Lightweight concrete can be used for bridge decks and prestressed concrete girders to reduce the weight that must be moved.

This paper discusses properties of lightweight concrete needed for bridge design. An example of the weight reduction for a bridge being moved is given. Several projects in Utah where lightweight concrete was used for the deck on bridges moved into place are highlighted.

INTRODUCTION

Construction of a bridge off-site and then moving it into final position by sliding or using self-propelled modular transporters (SPMTs) has become a more frequently used technique for ABC in recent years.

There are many benefits to this approach, including greatly reduced interruption of traffic, greatly reduced exposure of workers to traffic hazards, and improved quality of construction.

However, the movement of completed structures into place presents several design challenges, including the equipment required to lift and/or move the structure, and the way in which the structure must be supported during the move. In most cases, the structure must be supported inside the final bearing locations, which introduces overhangs and their associated negative moments which produce tension in the deck of the bridge.

Lightweight concrete can be used for bridge decks and prestressed concrete girders to reduce the weight of the bridge that must be moved. Lightweight concrete has been successfully used for bridge decks on many structures across the US. Its durability and strength has been demonstrated on bridges such as the

upper deck on the San Francisco-Oakland Bay Bridge suspension spans that was constructed in 1936 using all lightweight concrete (air dry unit weight of 95 pcf), and the deck is still in service today.

Lightweight concrete has been used less frequently for prestressed girders, although its use is becoming more frequent. A bridge in Florida constructed with a lightweight concrete deck and prestressed girders has been in service since 1964. Using lightweight concrete in a bridge reduces the negative moments that frequently result in cracks in the bridge deck during a move.

This paper briefly discusses the properties of lightweight concrete. An example of the weight reduction possible for a bridge moved by SPMTs is given. Projects in Utah where lightweight concrete was used for decks on bridges that were moved into place are highlighted.

PROPERTIES OF LIGHTWEIGHT CONCRETE

This section provides information on properties of lightweight concrete that is needed for bridge design. Earlier this year, the AASHTO Subcommittee on Bridges and Structures adopted some significant changes for the *AASHTO LRFD Bridge Design Specifications* related to lightweight concrete. One of the changes was the removal of the definitions of “all lightweight concrete” and “sand lightweight concrete” from the definition of lightweight concrete. Coupled with this change, the definition of the density modification factor, now identified as λ in the AASHTO LRFD specifications (as it has been known in ACI 318 for many years), is based on density rather than definitions that required a knowledge of the types of aggregate being used to make the lightweight concrete. These changes do not affect the outcome of designs using lightweight concrete.

Types of lightweight concrete

While the definitions of “all lightweight concrete” and “sand lightweight concrete” have been removed from the specifications, the definitions are still valid and are used in this discussion. Definitions for these terms follow, including a brief discussion of design compressive strengths and unit weights that can be achieved for each type of concrete.

All lightweight concrete: Lightweight concrete where all aggregate in the concrete mixture is lightweight aggregate. This type of concrete mixture provides the lowest possible unit weight for structural concrete. The minimum unit weight achievable depends on the characteristics of the lightweight aggregates used, and is typically about 95 to 100 pcf after drying, which is called the “equilibrium density.” The compressive strength that can be achieved for this type of concrete depends on the type of aggregate used,

but design compressive strengths of up to 6 ksi should be achievable for most types of lightweight aggregate.

Sand lightweight concrete: Lightweight concrete where the coarse aggregate is lightweight aggregate and the fine aggregate (sand) is normal weight aggregate. This type of concrete mixture provides a significant reduction in density from normal weight concrete and is the most widely used type of lightweight concrete. The typical unit weight for this type of concrete depends on the specified compressive strength and the characteristics and quantity of the lightweight aggregates used, and typically ranges from 110 to 125 pcf after drying. Design compressive strengths as high as 10 ksi have been successfully used for sand lightweight concrete. Higher compressive strengths typically require a higher density, as is also the case for normal weight concrete.

Other densities of concrete can be achieved by blending the different types of normal and lightweight aggregates that are available. In this way, any density from all lightweight concrete to normal weight concrete can be achieved. This is the concept of "***specified density concrete***" which generally refers to all types of reduced-density concrete containing lightweight aggregate. Using this concept, a designer or contractor can simply determine the concrete density needed to achieve a design or construction objective, and then specify that density, as long as the density is within the range that can be achieved using available materials. Lightweight aggregate suppliers and concrete suppliers should be consulted when considering use of lightweight concrete for a project.

It is also important to realize that lightweight aggregate is simply a lighter type of structural aggregate, so batching, placing and finishing are essentially the same as for normal weight aggregates used in normal weight concrete. Lightweight aggregate is typically prewetted prior to batching to satisfy the increased absorption of lightweight aggregates.

Modulus of elasticity

The modulus of elasticity of lightweight concrete is computed using the Equation 5.4.2.4-1 in the

AASHTO LRFD specifications, which includes a term for the unit weight of the concrete. Using the equation, which was updated in the 2015 interims to the AASHTO LRFD specifications, the modulus of elasticity is reduced as much as 50% for all lightweight concrete and from 65 to 75% for sand lightweight concrete mixtures.

Tensile strength

The tensile strength of lightweight concrete has long been assumed to be less than for normal weight concrete. As a result, the λ-factor was introduced to reduce factors related to the tensile strength of concrete when computing shear resistance, development lengths, and other quantities. Recently, it has been found that the tensile strength of lightweight concrete is often equal to or even exceeds the expected tensile strength of normal weight concrete. If this is the case, the designer can specify the splitting tensile strength of the lightweight concrete to be equal to the value expected for normal weight concrete, and the λ-factor will be unity, indicating no reduction in the computed quantities when compared to normal weight concrete of the same design compressive strength.

Creep and shrinkage

While creep and shrinkage of lightweight concrete has been assumed to be greater than normal weight concrete, recent research and testing of production concrete have shown that the values of these quantities for lightweight concrete are within the range of normal weight concrete. This allows designers to use equations for creep, shrinkage, and prestress losses that appear in the AASHTO LRFD specifications for lightweight concrete without modification.

EXAMPLE OF BRIDGE WEIGHT REDUCTION USING LIGHTWEIGHT CONCRETE

A comparison showing the differences between using normal weight concrete and lightweight concrete for the deck and prestressed concrete girders for a bridge to be moved by SPMTs appears in Table 1 (next page). This table is similar to the table that appears in Appendix C of the FHWA report "Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges." (1) The calculations are based on the Graves Ave. Bridge over I-4 in Florida which was one of the first bridges installed using SPMTs in the US. The bridge is 143 ft long and 59 ft wide and did not use lightweight concrete.

Table 1 differs from the table in Appendix C of the FHWA manual because it includes a third comparison for an all lightweight concrete deck and barrier design which provides a further reduction in weight of the structure. Data in the table show that the use of sand lightweight concrete for the girders, deck and barriers reduces the bridge weight by 233 tons, or just over 18%, compared to the normal weight concrete design. For the third case where all lightweight concrete is used for the deck and barriers and sand lightweight concrete is used for the girders, the weight is reduced 294 tons, or nearly 23%, compared to the normal weight concrete design.

The additional cost for using lightweight concrete in the bridge has been estimated using a typical cost premium for lightweight concrete mixtures compared to normal weight concrete mixtures. The cost

premiums for the lightweight concrete over the normal weight concrete will vary significantly depending on many factors, so this cost comparison is only for illustration and should not be expected to apply to all parts of the country. The cost for the lightweight concrete bridge alternate designs are between $2 and $3 per square foot of deck area more than the normal weight concrete design, which is a fairly modest increase in the cost of a typical bridge.

While an attempt has been made in the table to estimate the additional cost that can be attributed to the use of lightweight concrete in a bridge, other cost savings that result from the reduced superstructure weight have not been quantified. The reduced weight of the structure is expected to reduce costs for the following:

- temporary support structures,
- systems required to move the bridge, and
- reduced loads on permanent bearings and substructure units.

It is reasonable to expect that cost savings would be realized as a result of the approximately 20% reduction in superstructure weight indicated in the table by reducing the number of SPMT modules required to install the bridge or the size of equipment needed to mobilize and slide a bridge. The savings in these areas can possibly offset the extra cost for the lightweight concrete in the superstructure.

UTAH SPMT PROJECTS WITH LIGHTWEIGHT CONCRETE DECKS

Utah has been a leader in ABC projects. The initial bridges in Utah placed by SPMTs had normal weight concrete decks. However, the 3300 South Bridge over I-215, constructed in 2008, was the first bridge installed using SPMTs that was designed with a sand lightweight concrete deck. Inspections of the deck after a few years in service revealed that this bridge deck had less cracking than the normal weight concrete decks constructed and moved in the same or earlier years.

Table 1 Comparison of Total Bridge Weights and Cost Premium for Normal Weight and Lightweight Concrete Options

Total Weight and Estimated Costs for Normalweight and Lightweight Concrete Bridges

Bridge Information

Girder Type	FL Mod BT-78	
Girder Area	7.67	sq. ft
No. of Girders	8	
Length of Girders	143.00	ft
Total Deck Width	59.00	ft
Deck Thickness	8.00	in.
Haunch Thickness	3.00	in.

Compute Weight of Bridge

Girders	***NWC***		***Sand LWC***			***Sand LWC***		
Total Length	1,144.00	ft	1,144.00	ft		1,144.00	ft	
Conc. Density incl. Reinf	**152**	pcf	**127**	pcf	*83.6%*	**127**	pcf	*83.6%*
Total Weight	1,334,349	lbs	1,114,884	lbs		1,114,884	lbs	
Total Volume	325	cy	325	cy		325	cy	
Deck	***NWC***		***Sand LWC***			***All LWC***		
Total Volume	5,624.67	cf	5,624.67	cf		5,624.67	cf	
Conc. Density incl. Reinf	**150**	pcf	**120**	pcf	*80.0%*	**105**	pcf	*70.0%*
Total Weight	843,700	lbs	674,960	lbs	*168,740*	590,590	lbs	*253,110*
Total Volume	208	cy	208	cy		208	cy	
Haunch	***NWC***		***Sand LWC***			***All LWC***		
Total Volume	1,430.00	cf	1,430.00	cf		1,430.00	cf	
Conc. Density incl. Reinf	**150**	pcf	**120**	pcf		**105**	pcf	
Total Weight	214,500	lbs	171,600	lbs		150,150	lbs	
Total Volume	53	cy	53	cy		53	cy	
Barriers	***NWC***		***Sand LWC***			***All LWC***		
Total Wt / ft	1,200	plf	960	plf		840	plf	
Conc. Density incl. Reinf	**150**	pcf	**120**	pcf		**105**	pcf	
Total Weight	171,600	lbs	137,280	lbs		120,120	lbs	
Total Volume	42	cy	42	cy		42	cy	
Total Weight	***2,564,149***	***lbs***	***2,098,724***	***lbs***		***1,975,744***	***lbs***	
	1,282	***tons***	***1,049***	***tons***	***81.8%***	***988***	***tons***	***77.1%***
Weight Reduction			***233***	***tons***	***18.2%***	***294***	***tons***	***22.9%***

Estimated Cost Premium for Lightweight Concrete Bridge

			Assumed Cost Premiums for LWC						
Girders	325	cy	@	$	**30.00**	/ cy = $ 9,750	@	$	**30.00** / cy = $ 9,750
Deck, Haunch & Barriers	303	cy	@	$	**25.00**	/ cy = $ 7,575	@	$	**45.00** / cy = $ 13,635
Total	***628***	***cy***				***$ 17,325***			***$ 23,385***
Cost Premium / sf of Deck Area				$	***2.05***	***/ sf***		$	***2.77*** ***/ sf***

Notes:

NWC Deck & Girder mixes are assumed to have a plastic (fresh) density of 145 pcf with an allowance of 5 pcf for reinforcement in the deck and 7 pcf in the girder.

Sand LWC Deck mix is assumed to be 4.5 ksi with plastic (fresh) density of 115 pcf with an allowance of 5 pcf for reinforcement.

All LWC Deck mix is assumed to be 4.5 ksi with plastic (fresh) density of 100 pcf with an allowance of 5 pcf for reinforcement.

Sand LWC Girder mix is assumed to be 8 ksi with plastic (fresh) density of 120 pcf with an allowance of 7 pcf for reinforcement.

Sand LWC mixes use LW coarse aggregate and NW sand. All LWC mixes use LW aggregate for all aggregate in the mix.

Plastic (fresh) densities, rather than equilibrium densities, are used to compute loads for LWC. Time between casting and moving is likely too short for moisture loss and density reduction to occur.

Cost premiums for LWC mixes vary due to several factors, including distance to LW aggregate source and cost of NW aggregates.

Four bridges with lightweight concrete decks were installed with SPMTs in Utah in 2011, at least two of which were moved as 2 span structures (2). The bridges were:

- 200 South over I-15 – 2 spans @ 3.1 million lbs
- Sam White Lane over I-15 – 2 spans @ 3.8 million lbs
- I-15 Southbound over Provo Center Street - 2 moves of 1.5 and 1.4 million lbs
- Proctor Lane over I-15 – 2 spans set individually. Weight is not known.

The first three bridges were designed as steel girders with sand lightweight concrete decks, while the Proctor Lane Bridge uses precast prestressed concrete girders with a sand lightweight concrete deck.

The use of lightweight concrete decks for these structures reduces the weight of the structure to be moved, but also reduces the weight of the portion of the bridge overhanging the SPMTs. This is expected to reduce the cracking in the decks during movement of the bridges.

CONCLUDING REMARKS

Constructing bridges off-line and moving them into place is now being considered as a viable option by bridge designers across the nation, when circumstances are appropriate. Reducing the weight of a bridge that will be moved makes a lot of sense. Lightweight concrete has been used for years, so it is a reliable and durable solution for reducing the weight of a bridge to be moved, but will also reduce the load on the substructure units. This paper has provided a brief introduction to using lightweight concrete for these applications.

ACKNOWLEDGEMENT

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G-19: MISCELLANEOUS

THE FUTURE OF BRIDGE DESIGN

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INTRODUCTION

Design, modeling and analysis are keys to the success of today's bridge projects. The next generation of bridge modeling software requires that it be purpose-built for bridge designers and contractors who needto create, construct, maintain, and document a wide variety of bridge information throughout the lifecycle of the asset. Sharing information in an information-rich 3D model increases data quality, collaboration, constructability and operational aspects including asset management. Reduction in the project's overall costs for the entire ecosystem are important for all stakeholders and the availability of intelligent 3D models are a key component in providing accelerated project delivery and information mobility.

The purpose of this paper is to provide an overview of how technology can provide cost savings with the ability to interoperate with all stakeholders during design, construction and beyond on bridge projects of all sizes.

BRIDGE INFORMATION MODELING

The concept of BrIM (Bridge Information Modeling) has become more than a 'coined' term since introducing to the bridge sector. BrIM is an approach to intelligent bridge design, construction, project delivery, and operations/maintenance resulting in the creation of an information-rich data model that can be used throughout the life of the bridge asset. In just a matter of a few years, the industry has shifted focus to what it means to provide an intelligent and data-rich 3D model that connects design, construction, inspection, operations and maintenance. How much is too much information and to what is our goal? Can we shorten the construction schedule? Are we designing for construction and can we meet the expectations of the travelling public during construction with the proposed design? A true data model addresses these questions and progresses the integrity of our engineering profession. This is when vision meets reality.

Build it better trends – Driven to Innovate

Engineers, detailers and contractors are under increased pressures to find the solution that promotes better designs resulting in smarter and reliable construction methods. A disconnect between project and asset stakeholders is no longer acceptable in our industry and is driving the need for data interoperability throughout the lifecycle. Still imagery of pretty picture models doesn't lead to reality when constructability is at stake. The days of bidding on infrastructure projects has evolved into the owner/operator setting higher expectations for minimizing allowances for omissions or errors as added risks on the project.

This is leading us to a demand from the market for greater and better methods for reduction in construction costs (materials and time of construction) and minimizing the economic impacts. The MAP-21 compliance requirements, Everyday Counts (EDC) legislation and the growing popularity of design-build (DB) and Public Private Partnerships (P3) has set the stage for bridge project delivery expectations. These initiatives with design/construct/rehabilitate contracts are driving the owner demand for faster, more cost effective and constructible means of building our transportation assets.

Traditional Bridge Design Process

Unfortunately the bridge design and construction workflow is often a fragmented and linear process, with very little automation or exchange of data in a useful and integrated manner. The ability to reuse data across disciplines is challenging and it creates an environment prone to data re-input amongst multiple programs not to mention spreadsheets as a byproduct - resulting in the introduction of error- prone results. These processes traditionally involve centralized automation (roadway does roadway, bridge does bridge, inspectors do inspection) and there is minimal exchange of critical project and engineering data between key disciplines.

Picture a common scenario of designing a facility over a facility and all of the conflicts and challenges this presents. The instantaneous access and ability to tweak pier placement as you evaluate existing conditions is critical in these situations and common in design-build projects. Typically, geometric information transferred from the roadway design team is a manual and repetitive data entry process –change management likely does not exist in a manner that is efficient. There is simply no time to waste when alignments require shifting to keep a project moving forward. With errors, re-dos, model translations, multiple spreadsheets and shortened delivery timeframes, communication lines often breakdown while working under deadline pressures – data is dropped and most frequently not re-utilized.

In addition to the above challenges mentioned, the business case challenges are also immense. Dealing with the communication amongst teams and error prone workflows, or just dealing with multiple software products from different vendors can exasperate the disjointed design process. And lastly, but certainly one of the most important in our traditional project delivery is plans production. Often times this is not an automated process or at least not one that is either efficient nor automated in a manner that we are accustom to working as a direct result from our design and analysis tools.

Advancements in the Bridge Design Process

There are tremendous advantages in connecting the project team members with a 3D approach and technology. The benefits of having geometry that is relevant and the most current will tie the roadway and bridge engineers together from the onset of the project and throughout design revisions in a bi-directional manner. Not only are they working in a connected manner, they are working in a geospatially, for improved accuracy. Bridges can be developed and modeled in a true world manner and referencing existing conditions becomes easy and meaningful. Models can become the immediate mechanism for design and analytics. Imagine the time and cost savings of easily developing an intelligent model in the preliminary stages of a project – and carrying this through to design/analysis without the time or expense of reengineering. Most 3D modeling technology does not allow for a direct link to analytics without some reentry of data; nor do these models contain the level of detail required for today's projects.

The ability to link the physical model directly to the analytics would allow for alternate design options to be realized initially in the office as you are saving time and effort by previewing alternatives, constructability issues and conflicts in the earliest development of the bridge. Much of the design is for construction of course, but at what point do we begin to insert intelligence into the design so that we can predict and plan for construction? When does the design take into account construction steps? Today, that is a post design process, where the design is passed to a construction engineering team, who dissects the design and reassembles it into construction plans.

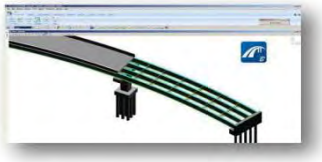
The question we should consider is, when does the modeler and engineer assume the responsibility of constructability and designing to build? A small tweak of geometry can determine where a skewed pier can impact potential field issues and can be tackled in the beginning of the project. With the massive amount of intelligence available between disciplines, it is only logical to collaborate and leverage it in a seamless workflow. We are here today with the industry solution and the tools available to make intelligent bridge design and analysis a reality. We need to take advantage of these tools and speed construction with little effort on the frontend. Small steps will ensure big

efficiency and accuracy as we advance through the information flow. We do it in manufacturing; we do it in civil construction. It is part of the evolution we must set as achievable goals to meet the industry demands for bridges.

From design/analytics, we can move to construction modeling – where is the crane location, can I maintain traffic? How am I going to build it?

An efficient bridge design process allows you to directly connect/reference both existing and proposed conditions/as well as civil data to perform constructability analysis – key to maintenance of traffic – facility over facility. By allowing you to visualize, render, perform clash detection, generate quantities and evaluate clearances with the information-rich model, you can be assured of reliable construction methods from the onset.

Bridge Modeling for Constructability and Beyond



Bridge modeling technology should provide engineers with an ability to create a workflow that promotes to true bridge information modeling and mobility. There needs to be an interoperability component that allows all project disciplines to evaluate and share critical data from the planning/bidding phases all the way through to commission, operations and maintenance. The true value of a model doesn't solely reside in its aesthetic appeal, but also in the usability and life of the data associated with it. Bentley's OpenBridge Modeler addresses the challenges we face with complex geometry needs, parametric updating of changes, and evaluating constructability early in the process as well as conflicts not seen in a 2D workflow. Easily work on a bridge project, share engineering rich data, and make more informed decision within a 3D model of the bridge project. OpenBridge Modeler, is the true gateway to BrIM – your key to today's design build projects, P3 and accelerated bridge construction.

The advantage of all disciplines (roadway, utilities, bridges, existing conditions, etc.) operating in a single modeling environment with no re-creation of critical project data is vital in meeting the challenges of the 3D deliverable by industry standards. OpenBridge Modeler provides a workflow specific to the needs of the bridge engineer –tools that model bridges – not buildings, yet facilitates collaboration and integration with other disciplines, such as civil engineers, utilities, and others to ensure everyone has the tools and data they need when they need it.

3D bridge models provide the ability to reference related designs that connect or affect the project. Clash detections with subsurface utilities, rebar detailing, bridge element placement, maintenance of traffic are

all key construction issues that in an integrated and interoperable workflow, can resolve upfront in the office rather than in the field – meeting the owner expectations for minimizing omissions and errors.

Summary

With a focus on operational excellence, sustainability and the economic impacts of a bridge not being available, it has never been more important to evaluate our processes. Bridge design and construction processes are evolving and 3D deliverables are imminent. Interoperability and collaboration are keys to the success of bridge projects of all sizes and construction methods. Leveraging complex geometry from the beginning to generate physical bridge models and preparing the design and analytical requirements is essential in moving to a more fluid and seamless reality modeling workflow. With intelligent As-Design Models and As-Built Data, engineers can provide operations and maintenance value for the entire life of the asset.

DECISION CRITERIA FOR DEPLOYING ACCELERATED BRIDGE CONSTRUCTION

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ABSTRACT

This paper presents an investigation of cost and risk factors that impact decision-making process for repair and replacement of structurally deficient bridges using prefabricated elements and advanced materials as part of accelerated bridge construction (ABC) technologies. The study seeks to create an

ABC resource database that a transportation agency can use to implement risk and lifecycle cost factors in decision-making.

INTRODUCTION

Public welfare, and socio-economic vitality in a modern society depend on the safety and functionality of civil infrastructure that includes buildings, bridges, roadways, lifeline systems (such as energy and water providers), and marine and airport structures. Within the transportation infrastructure bridges are essential links for communities, therefore disruptions to their operation often lead to congestion, safety concerns, increased travel time, and greater fuel consumption as the users are redirected toward alternate routes. In addition to the design and construction methods, multiple factors influence functionality and durability of bridges: traffic intensity, material degradation over time, environmental exposure, inspection intervals, damage detection capability, maintenance programs, as well as the extreme natural, or anthropogenic hazards. Because the bridge condition determines its structural behavior and serviceability, effective programs for assessment, maintenance, rehabilitation, and replacement of deficient systems are needed to extend the life expectancy of bridges, and to improve the functionality of the bridge network overall. Alleviating potential consequences of bridge deterioration and damage are especially important means of mitigating potential vulnerabilities of communities to the foreseeable multi-hazard events, when the safety of residents, emergency response, and recovery time depend on functionality of bridges and on the uninterrupted access to evacuation routes and critical healthcare facilities.

The challenges posed by deteriorating performance of the aging infrastructure combined with the awareness of limited resources for maintenance, upgrades, and replacement of inadequate structures have prompted a growing body of research related to risk-based approaches to evaluate prioritization scenarios, life-cycle costs, and benefits of the retrofit and replacement options. Risk-based methods have been used for decades in various fields including economics, finance, insurance, the energy sector, and medicine. The engineering disciplines focused on civil infrastructure have applied risk analysis in the recent decades to the development of performance-based assessment and design provisions for nuclear power plants, tall buildings, hospitals, and bridges (e.g., ASCE/SEI 31 [1], ASCE/SEI 41 [2], and ATC-63[3]).These guidelines have introduced predictive methods to estimate structural response to potential levels of seismic hazard based on the probability, risk, and inherent uncertainties that affect structural capacity of structures. A more complex approach, at a broader scale, involves evaluation of risk ensuing from hazards, structural vulnerabilities, and environmental stressors for systems that comprise a network of structures (such as buildings, or bridges), and also consider interdependencies between the social and physical systems within communities (McAllister [4], Moon et al. [5]).

Risk is commonly defined as a product of the likelihood that a hazard event will occur and the estimated cost of restoring the lost functionality in a system. Risk analysis methods have been extensively studied, and they include analytical modeling, expert-based assessment, and empirical methods. Analytical methods are based on rigorous probabilistic evaluation that involves reliability-based analyses, nonlinear finite element modeling, and simulations of load demand and system vulnerability to generate fragility curves which estimate the likelihood of structural damage (e.g., Nielson and DesRoches [6]). The complementary deterministic methods use empirical assessment of risk, based on historical data or on elicitation of expert opinion based on engineering experience (Saeedi et al. [7], Hallowell et al. [8]). In addition, risk-based decision making can be informed by physics-based models of material aging and decay, decision frameworks that are aimed at risk management of particular infrastructure facilities, or of specific risks, such as health and safety (Ellingwood [9]). Present report explores the synergies between risk based decision-making methods that are being implemented by state agencies in project management. By examining the risk factors that impact the allocation of funds for improvement of transportation infrastructure, project prioritization, project delivery methods, along with the developments in structural engineering that aim to increase construction efficiency and durability of bridges, the study explores present challenges and opportunities for implementation of accelerated bridge construction methods.

MOTIVATION FOR RISK BASED DECISION-MAKING

Destruction and damage of the infrastructure during the recent extreme events including the 2012 Superstorm Sandy, 2011 Tohoku Earthquake, 2010 Chile and Haiti Earthquakes, and 2005 Hurricane Katrina highlighted critical vulnerabilities of buildings, bridges and the lifeline infrastructure that provides distribution of energy and water. Direct losses due to the damage of structures, and the indirect economic and social losses, which were evident in the aftermath of these disasters, demonstrated the need to accelerate the upgrades and replacement of the aging urban infrastructure (Comerio [10], Furtado and Alipour [11]). The consequences of these events prompted the focus on developing guidelines that operationalize risk principles in engineering design and asset management practices within the state agencies that oversee inspection, maintenance and retrofit of the civil infrastructure. In the recent years, concurrent emergence of advanced computational tools that enable risk models and the standards recommending implementation of risk principles in engineering and management of infrastructure systems (MAP-21 [12], ISO [13]), have motivated transportation agencies to adopt risk analysis and performance-based decision making as part of the efforts to prioritize allocation of resources for upgrade and replacement of regional bridges. Prioritization of funds that are available for bridge construction is essentially an optimization process that aims to maximize the overall resilience of a transportation network while minimizing the cost, i.e., effectively allocating the available resources.

The 2012 report from National Academies (NAP [14]) defines resilience as “the ability to prepare and plan for, absorb, recover from, and more successfully adapt to adverse events”. The report suggests that “enhanced resilience allows better anticipation of disasters and better planning to reduce disaster losses rather than waiting for an event to occur and paying for it afterward.” As defined by Presidential Policy

Directive (PPD-21 [15]), resilience is “the ability to prepare for and adapt to changing conditions and withstand and recover rapidly from disruptions.” In one of the first conceptual frameworks that defined key dimensions of community resilience to seismic disasters, Bruneau et al. [16] proposed four “R”s, namely robustness, redundancy, resourcefulness, and rapidity. Robustness expresses remaining capacity of a system after it has been subjected to a specified level of load demand. Redundancy measures the potential for redistribution of the load carrying capacity among the system elements to maintain functionality. Resourcefulness is the ability to implement physical and technical resources to mitigate system disruption according to the prioritized goals. Rapidity distinguishes the methods that can be used to accelerate system upgrades before disruptive events occur, or as part of the recovery efforts that can be readily initiated in the aftermath of disasters.

Ability to implement, as standard practice, accelerated bridge construction (ABC) methods for retrofit or replacement of structures that are approaching a threshold of acceptable functionality is related to the resourcefulness and rapidity dimensions of the network resilience. ABC technologies are especially significant when we consider that the average age of U.S. bridges is 42 years, and that approximately 25% of the 610,700 bridges nationwide are designated as functionally obsolete or structurally deficient, requiring upgrades or replacement (NBI [17]). Thresholds of functionality for bridge structures are recorded in the National Bridge Inventory (NBI) database (USDOT [18]) and are defined using condition ratings for bridge elements. Drawing upon the resiliency definition by Bruneau et al. [16], Figure 1 illustrates hypothetical recovery scenarios to regain a desirable level of functionality in a bridge network, denoted by F_0. Earlier rehabilitation of deficient bridges, prior to reaching the functionality threshold F, and the use of rapid construction methods when appropriate, represented by the dashed curve. Such interventions can alleviate further deterioration, be less disruptive to the users and the commercial activity, and promote adoption of proactive preservation strategies that enhance community resilience. Condition of the infrastructure at the time of a hazard event affects the response of the system during the increased load demand due to the hazard. Therefore, mitigation of vulnerabilities prior to a disruptive event can reduce the damage, losses, and downtime of the infrastructure elements in the aftermath of the event.

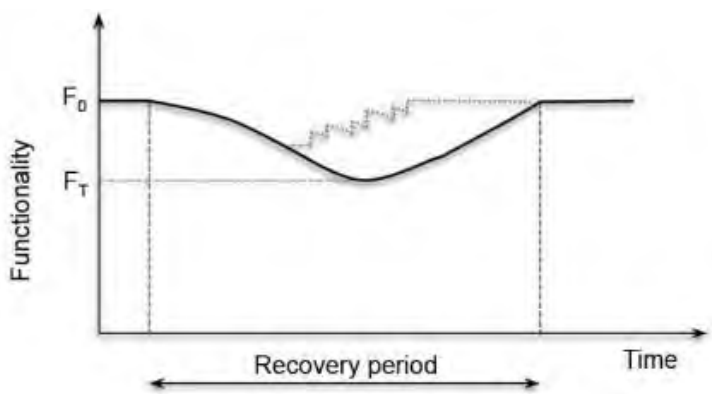

Figure 1. Relationship between bridge network resilience and rapidity of rehabilitation projects (Adapted from Bruneau et al. [16], and McAllister [4])

Transportation agencies can develop programs to implement risk-informed approaches that address multiple hazards, system vulnerabilities, performance thresholds, environmental exposure, and operational constraints. Primary goals of a risk framework are to: (1) provide qualitative-quantitative support in deciding among retrofit alternatives, (2) prioritize allocation of resources by balancing the needs of individual structures and those of the entire network, and (3) uncover interdependencies between the available resources, tools, methods, and policy that can be implemented into the system to reduce the

risks. As part of an agency's decision framework for planning bridge replacements or highway corridor improvements, the development of programs, standards, data, and documentation related to best ABC practices supports the goal of efficiency in construction and in the allocation of resources.

RISK MANAGEMENT INITIATIVES

Recent nationwide initiatives by the Federal Highway Administration (FHWA) express the vital need to implement proven technologies in rehabilitation of transportation facilities by "doing more with less."

Aiming to address the congestion and the bottleneck problem on the aging highways in urban areas, as well as the need to strengthen the infrastructure against disruptive hazard events, these initiatives support and mandate accelerated retrofit of deficient bridges, and performance-based management measures.

The Accelerated Technology Deployment initiative within Every Day Counts (EDC-1) emphasizes the need to optimize design and construction processes by the use of prefabricated elements and systems

(PBES). The FHWA EDC-2 initiative calls for innovations related to accelerated bridge construction

(ABC), which entail methods that range from building a bridge by using prefabricated or precast (PC) elements to deploying self-propelled modular transporters (SPMT) to move a bridge system from an offsite construction area to its final alignment in the structure.

MAP-21 legislation (MAP-21 [12]) builds upon the EDC initiatives by requiring the use of metrics tied to safety and condition of the infrastructure for decision making and optimizing the use of resources. The amendments to the National Highway Performance Program (NHPP) mandate development of a "risk based asset management plan for the National Highway System" (NHS) to provide support for assessing performance and construction of new facilities in the NHS. The contents of the performance driven management plan includes: (a) a summary listing of the pavement and bridge assets on the National Highway System in the State, including a description of the condition of those assets; (b) asset management objectives and measures; (c) performance gap identification; (d) lifecycle cost and risk management analysis; (e) financial plan; and (f) investment strategies. The Program stipulates a potential 35% reduction in Federal funding for States that do not implement such a management plan (MAP-21 [12], 1106, Sec. 119(e)(5)).

A number of transportation agencies have recently adopted frameworks based on risk associated with particular facilities (e.g., bridges, or pipelines), and for certain types of hazards (e.g., seismic, or flooding) that are considered to be significant due to foreseeable safety concerns and related consequences for the

region.The expanded awareness of risk exists in the departments of transportation (DOTs) as their managers face increasing demands on the aging infrastructure, and the mandate to operationalize decision-making based on quantitative models for the entire organization (e.g., for a population of bridges) in addition to performance-management of individual projects. There is an emerging recognition that the enterprise risk management (ERM) practice, which has successfully been adopted in the private sector in the last decade, can just as effectively be applied in the public agencies. ERM is an approach by which an entity manages “all risks viewed together within a coordinated and strategic framework” as opposed to managing independently one risk at a time, which is also known as the “silo method” (Nocco and Stulz [19]). ERM is a process that evolves over time within an organization, is based on evaluating interdependencies of risks and on strategic decision-making to achieve long-term goals of the enterprise.

Benefits of ERM include more objective allocation of resources, reduced volatility, development of a more informed and transparent culture of risk management, ability to convey the risks to the stakeholders, and increased stakeholder engagement and confidence.

At this time ERM is in the early stages of implementation in the private industry, therefore case studies are mainly used to demonstrate its potential applications and effectiveness (Gates [20]). A recent comprehensive survey of risk-management practices in the U.S. departments of transportation (DOTs) reveals present interest, opportunities, and hurdles in formalizing the ERM programs within the transportation agencies (Hallowell et al. [8]). Based on the responses that the study received from 43 out of the 52 surveyed DOTs, 81% of the respondents stated that formal, published risk management policies were in place at the project or program level, while 39% of the DOTs had implemented an ERM program.

Risk management at the program level involves decisions that impact groups of projects. For example, a revision of agency’s asset preservation guidelines represents a programmatic decision that aims to extend the service life of assets in several types of projects. Project-level risk management addresses risks that are related to a specific retrofit or maintenance project. In addition, 40% of the respondents expressed the belief that proper risk strategies were seldom or never implemented. Finally, while the commitment of upper level managers to integrating risk policies throughout the agency was deemed to be essential for successful implementation of ERM, the study found that engaging more employees can add valuable, diverse perspectives for identifying risks and developing an appropriate decision framework. This approach also serves to improve the general understanding of risk-based decisions and their potential effect on the entire organization.

ABC INITIATIVES WITHIN STATE AGENCIES

Expanded implementation of accelerated bridge construction (ABC) methods in the recent years has demonstrated their potential to reduce traffic delays and congestion while the bridge is out of service due to construction activities, to increase work zone safety by shortening the exposure of construction workers to the moving vehicles and roadway hazards, and to improve quality control and durability for bridge elements that are cast off-site. While these and other advantages of the ABC methods have become evident and recognized, areas on which states seeking to develop ABC experience and expertise can place emphasis are screening and assessment of candidate projects for ABC, standardization of technology solutions to achieve economy of scale, documentation of lessons learned, and effective communication with stakeholders related to risk and life-cycle cost (versus “first cost”) considerations.

The following sections will discuss these areas in the context of risk management, by outlining how ABC methods can facilitate improvements in the resilience of transportation infrastructure, and by addressing the risks related to ABC implementation.

Screening and Assessment of Candidate Projects for ABC

Departments of transportation can use AASHTOWare Bridge Management software BrM (AASHTOWare [21]) to support data storage for the bridge inventory and data analysis. As part of a state agency Bridge

Management System (BMS) the software aims to provide data analysis capabilities that bridge owners can use to estimate the agency’s structural needs based on performance indices, and to recommend preservation, rehabilitation, or replacement actions. Potential uses include development of so-called “benefit group impacts” which allow screening of the effects that specific actions may have on groups of bridges with common characteristics (e.g., all concrete or steel girder bridges) that an agency oversees, which can be a powerful planning support tool. BrM (formerly Pontis) is currently used to store bridge inspection data such as condition assessments that are based on National Bridge Element (NBE) ratings. Condition state 1 (CS1) indicates “good” condition of an element. CS2 corresponds to a “fair” condition in which first signs of deterioration appear. For example, minor delamination and spalling of concrete, or freckled rust in steel elements may be seen. CS3 denotes “poor” condition of an element in which deterioration has progressed, and may include large concrete spalls, loss of section, cracks, missing steel bolts and fasteners, and exceeded tolerable limits of deformation. CS4 is defined as “severe” condition that warrants structural evaluation of the element or the system.

Figure 2 illustrates an expert elicitation process conducted by Bentley Systems to configure state-specific deterioration models for a future version of the BrM software. The experts in a state agency provide experience-based opinion regarding element deterioration over time, assuming that no maintenance actions are taken. The discrete data points denoted as Expert CS1, indicate percent area (or length) of an element that is expected to reach the deterioration level equivalent to condition state 1 (CS1) in a given year. Similarly, the Expert data points for CS2, CS3, and CS4 are obtained.

Relative weights are assigned to the percentage indicated for any year as a measure of confidence that experts have in their estimates. For example, greater confidence of opinion about element deterioration between the years 0 and 15 is indicated by the 100% relative weight assigned to the data points for that range of years. A reduced confidence in the anticipated condition of the same bridge element at 50 years of age is expressed by assigning a 25% relative weight to the related data point. Therefore, by assigning a smaller relative weight to the opinion about older elements, this data bears less influence on the resulting deterioration curves than the data for the earlier years in the service life of a bridge (Bentley Systems [22]).

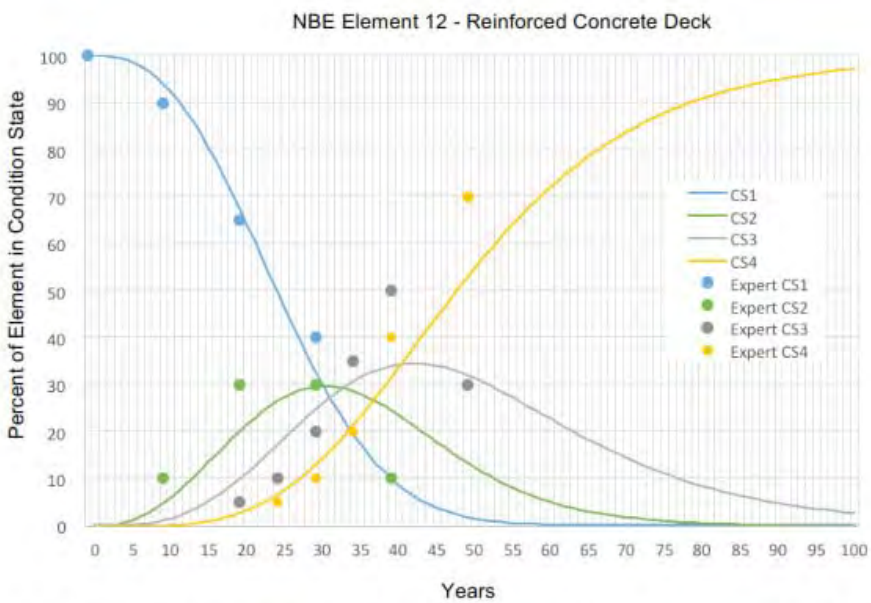

Figure 2. Deterioration data elicitation example for NBE element 12, reinforced concrete deck (Bentley Systems 2015)

Such data elicitation reveals and quantifies experiential knowledge about performance of particular bridge elements, systems, and component materials, such as steel, reinforced concrete, or prestressed concrete. In addition to this data, specific information such as route criticality, historic designation, scour critical designation, bearing conditions, distance of detour routes, network redundancy, average daily traffic,

maintenance cost, replacement cost, would be incorporated in the BrM database and available to inform project selection decisions and also the potential merits of using ABC methods for design and project delivery. In general, data collection and storage using data management tools can provide unique capabilities and platform for a holistic analysis of the bridge network across multitude criteria, for various hazard types, vulnerabilities of structural systems, details, and interdependencies that may lead to cascading disruptions within the network, or provide insight on impactful rehabilitation strategies by relating consequences or costs with particular options and decisions.

As part of an enterprise risk management (ERM) approach, risk-based project prioritization framework for a state agency can be structured to provide a greater level of detail in areas where decision-making requires higher-resolution data than the current designations of structural deficiency and functional obsolescence allow (Moon et al. [5]). An objective, quantitative risk-based method can be used to comprehensively evaluate the likelihood of hazards that would mobilize particular structural vulnerabilities, resulting in consequences if bridge elements reach particular limit states. Considering that risk-based prioritization of bridges relies on data mining and analyses that are also needed to determine whether the ABC methods are more effective than the conventional construction for a particular project, synergies between these efforts can serve as the basis for formalizing the project screening for ABC.

Table 1 lists relevant decision criteria, or risk parameters, that have been evaluated qualitatively and quantitatively by past research focused on risk assessment for infrastructure improvement (e.g., Nielson & DesRoches [6], Hallowell et al. [8], NAP [14]), and also on development of ABC decision-making tools (Saeedi et al. [7], Furtado and Alipour [11], Gannett Fleming [23]). The risk parameters can be quantified using analytical models, expert elicitation, or databases such as the NBI [17]. For example, direct cost associated with replacement of a bridge structure can be estimated by multiplying the area of the bridge by the cost per unit area of the bridge. Indirect costs, such as those imposed on the drivers due to the construction-related delays, or loss of business opportunity, can be calculated using analytical models (e.g., Furtado and Alipour [11]). Certain parameters, such as the presence of “fracture critical details” are described qualitatively, using “yes” or “no”, according to the NBI code [17]. Based on past research, case studies, and expert opinion, risk values are assigned to the possible parameter values, and aggregate risk score is obtained after weighting of risk factors. While the process of determining the appropriate project delivery method (ABC versus conventional construction) pertains to a single project, and has specific requirements and comparison aspects that may not impact the network project prioritization, these two processes are closely intertwined due to the common risks that they evaluate. In addition, state-specific programs for ABC implementation may require network-wide assessment in addition to project-level decision making. Likewise, a database of risk and cost factors that determine effectiveness of ABC

implementation can inform the capital planning process aimed at improvements of infrastructure resiliency.

Table 1. Risk and life-cycle considerations for project prioritization and ABC decision making

Load Demand/Hazard	Structural Vulnerability	Consequence	Constructability	Durability
Flooding	Ductility	Safety	Site access/ Right of way	Life cycle cost
Seismic	Overload	Network redundancy	Utility/ Railroad impacts	Material selection
Vehicle/vessel impact	Redundancy	Route criticality	Water crossing	Structural system
Average daily traffic	Fatigue/fracture critical	Delay/detour time	Geometry/ alignment constraints	CIP/prefabrication quality
Freight load	Overtopping	Traffic impact	Weather limitations	Connection details
Weather/climate	Scour critical	Construction duration	Repetition/ Economy of scale	Ease of inspection and maintenance
	Soil characteristics	Replacement/ maintenance cost	Expertise	Chemical/ environmental attack
		Delay related road user cost	Environmental impact (site/habitat preservation, air quality, noise)	
		Social impact/ loss of business income	Corridor improvement	
		Toll revenue loss		
		MOT		
		Public perception		

Standardization of ABC Methods for Durability

Advancements in ABC systems are motivated by the benefits of minimizing the on-site construction activities and development of durable solutions for bridge structures. For example, through the use of prefabricated elements, field-casting of concrete can be minimized, thereby alleviating the extensive traffic delays associated with cast-in-place construction, the impacts of construction on the site, and disruption to the environment. Except at connections and at concrete element closures, prefabricated components require no filed-casting of concrete for structural elements, and therefore offer potential for greater quality control of fabrication and durability. Tighter tolerances and less variation due to shrinkage and temperature changes during the initial curing of concrete can be achieved for plant-produced components.

While ABC techniques have been employed in a variety of superstructure and substructure systems, development of ABC methods for bridge decks will be discussed within the scope of the present report, since deck repair and replacement account for more than 80% of the federal and state resources allocated

for maintaining the U.S. bridges. Current practice assumes the bridge service life to be 75 years. Considering that a typical bridge deck service life equals approximately 25 years, and that through the leaking deck joints aggressive deicing chemicals can be carried through to the superstructure, caps, and columns, durability of bridge decks is an evident area of opportunity for improvement. Key factors that affect deck durability are permeability of concrete, and corrosion of reinforcement, the latter being one of the main influences on initiation of cracking and eventual spalling of concrete.

Low permeability of high performance concrete has been demonstrated in experimental tests and fieldapplications, however, the use of non-shrinking grouts and ultra-high performance cement composites in the precast panel shear connectors represents the weakest link for both deck durability and composite action. Past research and observed behavior of bridges using PBES indicates that durability of these systems greatly depends on the performance of joints (Gannett and Fleming [23], Szary et al. [24]). The consistent concrete cover of reinforcement in the prefabricated panels provides added protection against initiation of rebar corrosion, however quantifying durability gains and the associated life-cycle benefits of these systems can be challenging when collection of field data takes decades.

The new Bridge Evaluation and Accelerated Structural Testing (BEAST) laboratory at Rutgers University will enable scientific study and quantification of decades-long deterioration by performing “timecompressed”, realistic simulations of in-service conditions emulating the environmental, traffic, and chemical stressors on typical full-scale bridges. In the enclosed environmental chamber seen in Figure 3, bridges can be subjected to rapid cycles of controlled freeze-thaw temperature variation between 0°F 104°F, precipitation, deicing chemicals, and 60 kip load applied through a truck chassis that is travelling continuously during the course of the experiment at a speed of 20 mph. Such accelerated bridge aging is expected to reveal the patterns of decaying effects after 15-20 years of service life within several months of simulations.

Experimental results obtained in the BEAST lab can be used to develop guidelines for implementation of ABC technologies based on the ability to concurrently subject to the same conditions and test several types of precast and cast in place bridge deck panels. Present areas of concern related to PBES, such as durability and life-cycle costs, can be addressed. Influence of creep on long-term performance, and of shrinkage on joint movement, cracking, and liquid permeation, would inform the current practice of transporting and installing precast slabs within the initial 6 month period following the casting, when slabs typically undergo considerable shrinkage. Alternatives for panel-to-panel and panel-superstructure connections, advanced materials, and configurations for concrete closure pours can be evaluated to identify successful applications. Economical and field-usable methods for leveling the These studies

would support development of ABC standards and specifications, which are often cited by state agencies to be lacking.

(a) Simulation of the bridge deck installation at the laboratory

(b) Lab construction site during placement of the environmental chamber

Figure 3. Bridge Evaluation and Accelerated Structural Testing (BEAST) laboratory at Rutgers University

SUMMARY

This report presents an overview of risk analysis approaches that are based on analytical modeling, expert-based assessment, and empirical methods. In the recent decades risk-based methods have been implemented in the development of performance-based assessment and design provisions for power plants, buildings, and bridges. A broader risk assessment approach involves analysis of a network of structures, or a community, to evaluate the risk level ensuing from hazards and structural vulnerabilities while considering interdependencies between the social and physical aspects of the system. The role of

ABC technologies in improving infrastructure resiliency by restoring system functionality within a reduced time period was discussed. Drawing upon current development of tools used for bridge management systems, opportunities to introduce in these tools the methodologies for risk-based prioritization of projects and project screening for ABC implementation are discussed. In addition,

improvements in materials and joint configuration for ABC projects were discussed in the context of the new lab facility for accelerated testing of full-scale bridges.

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200+ YEARS OF ACCELERATED BRIDGE CONSTRUCTION IN AMERICA

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INTRODUCTION

Before there were smart phones and wireless internet, the greatest innovators in the world, were bridge engineers. Civil engineers and more specifically, bridge engineers, literally ushered in the "Iron Age" and later the "Steel Age" with innovative bridges. Further, every great paradigm shift in the bridge world originated from a patented idea or "Intellectual Property", generally marketed as a proprietary product. "The bucolic covered bridge; the ethereal appearance of prefabricated metal trusses thrown across numerous streams; traditional arch and girder forms appearing in the garb of a new material, reinforced concrete – these altogether elicited more than eight hundred patents during the first century of the U.S. Patent Office." (Kemp, 2005) Even newer advances including suspension bridge technology, movable bridges, prestressing and post-tensioning, like reinforced concrete were all the result of patented intellectual properties.

Our heritage as engineer-inventors is still alive and a new class of structures and bridge products continues to emerge, many using advanced high-performance materials. Although not as ubiquitous as the "Catalogue Bridges" that shaped the industry and the American landscape for nearly a century after the Civil War, these new technologies offer the same opportunities for a paradigm shift that will positively impact the quality of the built environment. This presentation will focus on an overview of two centuries of historical and patented innovations that ushered in the origins of ABC dating back to the infancy of our country. The discussion will demonstrate the catalysts that have continually driven bridge engineers to advance the state of our industry.

HISTORY OF PATENTED BRIDGE TECHNOLOGY

At the advent of the formation of the United States the founding fathers recognized the value of allowing its citizens to benefit from the promotion of intellectual properties. The primary champion of this cause was Thomas Jefferson. Jefferson initially opposed patents strongly because he considered it an unfair monopoly. He would later become more in their favor when he discovered the power they had to encourage invention. For Jefferson the purpose of the patent office was to promulgate invention, not

protect them. These two reasons are why he formulated a policy for patents that encouraged invention but maintained restrictions on what could be patented. Thus he was able to be true to his beliefs and perform the duties foisted upon him by the Patent Act of 1790 (Jewett 1994).

The legal basis for the United States patent system is Article 1, Section 8, of the United States Constitution wherein the powers of Congress are defined. It states, in part:

"The Congress shall have Power...To promote the Progress of Science and useful Arts, by securing for limited Times to Authors and Inventors the exclusive Right to their respective Writings and Discoveries".

With this simple act, the United States Congress avoided one of the greatest barriers to innovation by incentivizing the commercialization of intellectual properties by any inventor willing to expend the effort and take the risks of introducing a novel idea. In the first century of the United States, this led to the development of over eight hundred patents related to advances in bridge technology.

In the early years the United States was still an agrarian society and the industrial revolution had not yet begun to take place, regardless it became evident that a robust transportation network would be necessary in a country that was still comprised of mostly wilderness. At this time there was no such thing as harnessing electricity or nuclear power and even chemical and mechanical engineering were essentially non-existent. The earliest engineering discipline was essentially Civil Engineering and in particular, bridge engineers began to emerge as the rock stars of this developing nation.

The Evolution of Truss Bridges

In the late 1700's and early 1800's constructing bridges out of metal was not really an option and the idea of reinforced concrete had not yet been invented. In older civilizations the earliest bridges had evolved as stone arches. But in the new world, there were insufficient masons to construct as many arch bridges as would have been required for such rapid growth. To build more bridges, faster and over longer distances, the most readily available materials and craftsmen favored construction out of timber. Further, there were few engineering colleges, much less skilled bridge engineers to design the number of bridges required. The result is that small contingent of skilled craftsmen with some understanding of statics and mechanics of materials developed wooden truss systems to facilitate more rapid construction of bridges. The earliest versions of these included the "Arch Truss Bridges" developed by Theodore Burr. Others who helped foster in this age of patented wooden truss systems included Lewis Wernwag, Stephen Harriman Long and Ithiel Town.

As cast iron and wrought iron began to emerge as more economical and readily available materials, other inventors began adding to the suite of bridge solutions through patented truss designs comprising hybrids of wood, cast iron and wrought iron. The earliest of these included William Howe as well as the father and son team of Thomas and Caleb Pratt. The Howe Truss and Pratt Truss are still ubiquitous terms used today to define the specific geometry and arrangements of the members of the respective bridge systems developed by these gentlemen. It is these original structural systems that ushered in the first series of "Catalogue Bridges" that allowed railroads and municipalities to expedite the construction of bridges by simply ordering a standard, pre-engineered bridge custom fabricated to fit a particular site.

Numerous other innovated structural systems followed and ultimately provided a plethora of patented bridge systems that included truss systems known as; Howe, Pratt, Bollman, Fink, Whipple, Warren and Petit Trusses, to name a few. As the US Patent and Trade office limits the duration of utility patents, there was always an incentive to find new and better solutions to help drive the evolution of bridge technology, while granting the inventors of these systems the right to capitalize off of their intellect.

The Impact of the Railroad Industry

With the advent of the Iron Horse, the demands for rapid construction of bridges became even more prevalent. The railroads predated the automobile by more than seven decades and the rapid expansion of the intricate network of railroads throughout the country demanded standardization and expediting of bridge fabrication and construction. Although the history books typically focus on the monumental bridges of this period like the Eads Bridge and the Brooklyn Bridge, these only represented a very small portion of the population of bridges being constructed. In particular, after the end of the Civil War the rapid expansion to the west represented an unprecedented period of growth. As noted in his book "American Bridge Patents", Emory Kemp points out:

> "Thus, "catalogue bridges" appeared on the scene after the Civil War and became a ubiquitous feature on the American landscape. Arguably, they had a more profound effect on the development of the American highway system than the justly famous landmark bridges by luminaries such as John A. Roebling, James B. Eads, Theodore Cooper, or James Finley."

The demand for these types of bridges fostered a host of companies dedicated specifically to providing rapid fabrication and delivery of pre-engineered bridges such as; Keystone Bridge, American Bridge, Chicago Bridge & Iron, Phoenix and King Bridge Companies. Many of these companies have since fallen by the wayside or succumbed to consolidation. Although American Bridge and Chicago Bridge & Iron still exist to this day, neither continues to market and sell proprietary "catalogue bridge" solutions.

The railroads also pioneered the concept of “Accelerated Bridge Construction” as a basic necessity to maintaining revenue service. To this day, very few railroad bridges are afforded the luxury of reconstruction with a complete shut down of traffic for more than eight hours. This prompted the railroad bridge engineers to develop “shoe flies” as a method of constructing temporary bridges on a slightly offset alignment to facilitate replacements. They also pioneered the bridge slides and prefabricated bridge elements that allowed for rapid construction with rail mounted cranes. Many of the techniques now being developed for highway bridges originated with the railroads decades earlier.

OTHER BRIDGE ADVANCES FOSTERED BY INTELLECTUAL PROPERTY

In addition to the patented truss systems, almost every advance in bridge construction was driven by the incentives of intellectual properties. Almost every movable bridge system developed in the late 1800’ss and early 1900’s was a product of IP, including the Chicago Bascule developed by Joseph Strauss, the Scherzer Rolling Lift and Rall Bridge developed by Joseph Rall. Patents also drove the evolution of suspension bridge technology.

Reinforced concrete itself was a patented technology. Ultimately this led to the development of post-tensioning systems such as the one developed by Eugene Freyssinet. This ultimately led to the development of post-tensioned segmental construction by Jean Muller in the late 1950’s. In fact the first post-tensioned segmental bridge comprised a single span bridge in Wyoming County, NY. In order to facilitate rapid construction in a site location that required rapid construction, Muller fabricated the span out of three smaller pieces of concrete with transverse, match-cast joints which allowed for trucking to the site, setting in place and post-tensioning together in very short construction window. Subsequent to the catalogue bridges, this represented one of the first “Prefabricated Bridge Elements Systems” to come along in decades.

WHERE WE ARE NOW

The imaginations of bridge engineers have not diminished with time. Over the past decade in particular there has been a re-emergence in the need to develop ABC technologies to placate the impatience of the travelling public. The additional costs for ABC can often be justified by the impacts to the traveling public and local economy. The emergence of SMPT’s, prefabricated bridge elements and ultra-high performance concretes have helped foster a new found interest in developing innovative solutions.

Even prior to this period several innovative structural systems managed to find their way to market in the last thirty years, including Inverset, ConSpan and Reinforced Earth retaining wall systems. All of these represent clever systems that lend themselves to standardization and ABC. All of them have evolved as a

result of intellectual properties. Regardless, these new age "catalogue bridge" technologies have had additional hurdles to overcome to gain widespread acceptance. The most challenging of these is Title 23 CFR 635.411 that was signed into law in 1976 prohibiting the use of proprietary products to be used on federally funded projects with but a few exceptions. Since this time, the barriers to innovation in transportation infrastructure have led to stagnation in development as a result of the loss of a very important incentive.

In conclusion, to gain a better appreciation for the recent developments in ABC it is important to recognize the technologies developed over the last 200 years in America that led to many of the systems used today. Further it is important to recognize the role that patents and intellectual properties played in the acceleration of these systems since the beginning of our nation. An opportunity exists with the inherent benefits of ABC to revert the culture of the civil engineering profession back to one of innovation.

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SEISMIC BEHAVIOR OF PRECAST HOLLOW-CORE COLUMNS FOR ACCELERATED BRIDGE CONSTRUCTION

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ABSTRACT

This study presents the structural behavior of precast hollow-core fiber reinforced polymer-concrete-steel (HC-FCS) bridge columns under combined axial and lateral loading. The HC-FCS column consisted of a concrete wall sandwiched between an outer fiber reinforced polymer (FRP) tube and an inner hollow steel tube. Two large scale columns, including one conventionally reinforced concrete (RC) column and one HC-FCS column, were investigated during this study. Each column has an outer diameter of 24 inch. and an aspect ratio (height-to-diameter ratio) was 4.0. The steel tube was extended inside the footing with an embedded length of 1.6 times the steel tube diameter. The FRP tube only confined the concrete wall thickness and stopped at the top of the footing level. The hollow steel tube was the only reinforcement for shear and flexure inside the HC-FCS column. In general, the HC-FCS columns exhibited a good behavior under seismic loading with a high lateral drift reached to 15.2%. The RC-column failed at drift of 10.9%. The RC-column failed by rebar fracture and the moment capacity suddenly dropped more than 20% after that. However, the HC-FCS failed gradually before the FRP rupture.

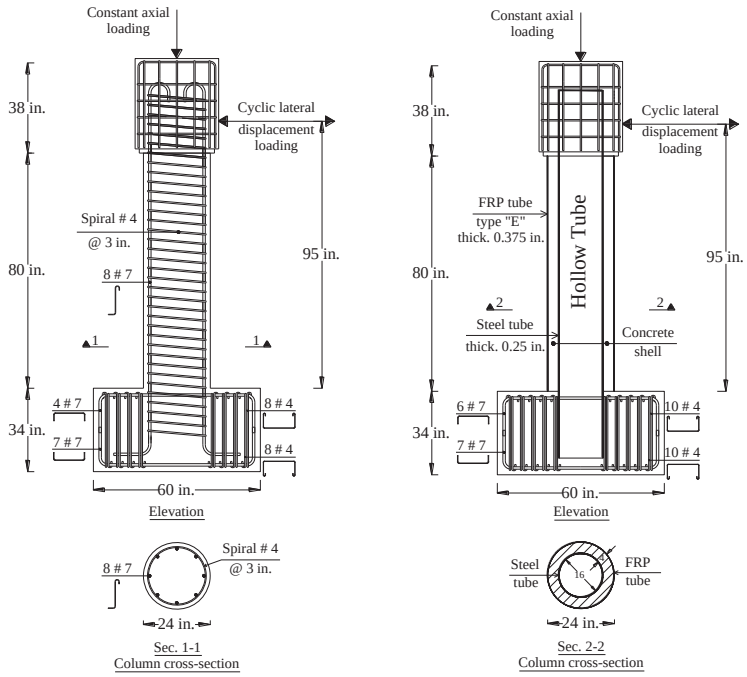
Introduction

Recently, several research groups have devoted their studies to develop new materials and construction methods for cost-effective accelerating bridge construction (ABC) systems. The ABC systems improve site constructability, reduce total project delivery time, enhance work zone safety for the traveling public, reduce traffic disruptions, and reduce life-cycle costs [1, 2]. This study investigated the behavior of an innovative precast hollow-core fiber reinforced polymer (FRP)-concrete-steel (HC-FCS) bridge columns in large-scale under combined constant axial load and lateral cyclic loading comparable to the behavior of the reinforced concrete (RC) columns. The HC-FCS column consists of a concrete wall sandwiched

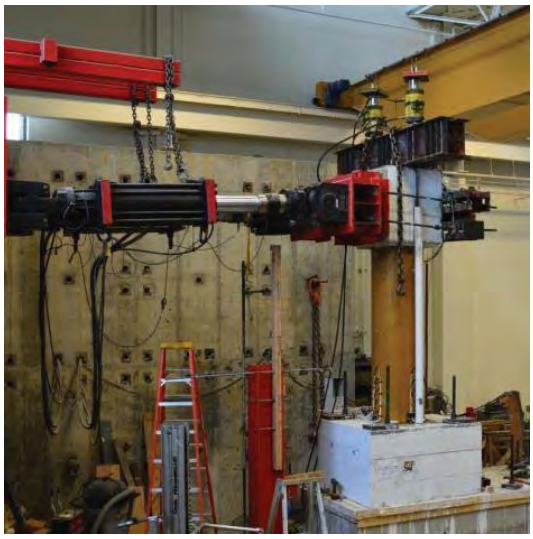
between an outer FRP tube and an inner steel tube. The steel tube of the HC-FCS column is extended inside the footing with a certain embedded length (L_e). The FRP tube only confines the concrete wall thickness and stops at the top of footing. The HC-FCS column has several distinct advantages over the conventional column constructed out of reinforced concrete (RC). The HC-FCS column uses 60 to 75% less concrete material since it has a hollow-core. Constructing a HC-FCS column in the laboratory during the course of this study required 90% less construction time. The HC-FCS column also requires a lower freight cost when implemented with precast construction. The HC-FCS column represents a compact engineering system; the steel and FRP tubes act together as stay-in-place formworks and the steel tube acts as a flexural and shear reinforcement. The inner steel and outer FRP tubes provide a continuous confinement for the concrete shell; hence, the concrete shell achieves significantly higher strain, strength, and ductility compared to the concrete of the conventional RC column. Due to the protection afforded by the corrosion-free outer FRP tube and concrete core, the HC-FCS column has a high corrosion resistance. In addition, hollow-core columns reduce the column's mass which reduces the bridge self-weight contribution to the inertial force during an earthquake. Hollow-core columns also result in reduced foundation dimensions which reduce substantially the construction cost.

EXPERIMENTAL PROGRAM

Two large-scale columns were tested as free cantilevers under both constant axial compression load and cyclic lateral load. Each column had a circular cross-section with an outer diameter (D_o) of 24 inch and a height of 80 inch (Figure 1). The lateral load was applied at a height (H) of 95 inch measured from the top of the footing resulting in shear-span-to-depth ratio of approximately 4.0. The first column was a conventional reinforced concrete (RC) column and the other column was HC-FCS column.



(a) (b)

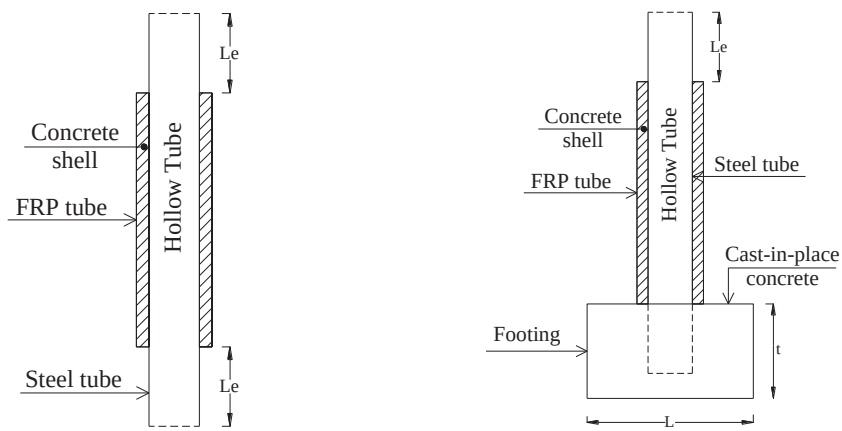


(c)

Figure 1: Tested columns: (a) Details of RC column, (b) Details of a HC-FCS column, (c) Test setup

Construction sequence

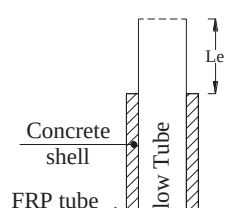
There are two options to construct the HC-FCS columns. The first option; building the precast HC-FCS column in the precast yard, then install it on the reinforcement cage of the footing, and finally cast-in-place the concrete footing (Figure 2). The second option; building the precast HC-FCS column in the precast yard during casting-in-place of the footing with a certain void, then install the precast column into the footing's void, and finally grouting the gap between the footing and the steel tube (Figure 3). The void diameter (D_o') is larger than the column's diameter (D_o) to free access for grouting. Option "1" has lower number of tasks for construction but the tasks are series while option "2" has higher number of tasks for construction but the tasks are parallel.

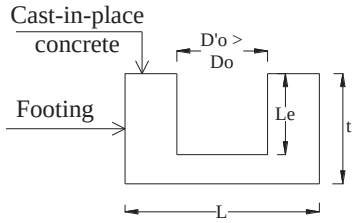


Step 1: Build pre-cast HC-FCS column

Step 2: Install pre-cast column on the footing cage then cast-in-place the footing

Figure 2: Construction of HC-FCS column: Option "1"





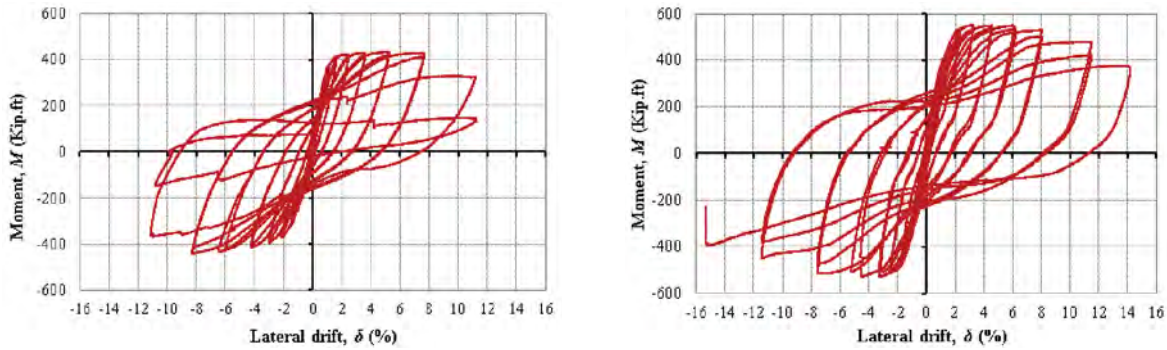
Step 2: Cast-in-place the footing with void of dimensions D'_o x L_e

Step 3: Install pre-cast column on the footing's void then concrete grouting to fill the gap between the steel tube and the footing

Figure 3: Construction of HC-FCS column: Option "2"

Results and discussions

Figure 4 illustrates the moment-lateral drift relation of the RC and the HC-FCS columns. The lateral drift (δ) of each column was obtained by dividing the lateral displacement measured from the actuators and corrected for any footing sliding, by the column's height of 95 inch The moment (M) at the base of the column was obtained by multiplying the force collected from the actuators' loading cells by the column's height of 95 inch. As shown in Figure 4(a), the average peak moment capacity of the column RC was 438 kip.ft at lateral drift of 5.1%. The stiffness of the column displayed gradual stiffness degradation up to a lateral drift of approximately 2.0%. Beyond that drift, significant stiffness softening started. The failure of the column occurred at lateral drift approximately 10.9% due to fracture of two rebars at the north and south side of the column. Failure was defined as the column loses at least 20% of its flexural capacity. For the HC-FCS column, the average moment capacity of the column was 540 kip.ft at lateral drift of 2.8% (Figure 4(b)). Gradual stiffness degradation occurred until drift of 2%; beyond that the column suffered significant stiffness softening. The peak lateral strength of the column remained approximately constant till a drift of approximately 6.0%. Beyond that, the column's flexural strength gradually decreased till drift of 10.9%. At this drift ratio the column suffered 11% strength reduction. Cycling continued beyond that and the FRP ruptured at lateral drift of 15.2%



(a) *(b)*

Figure 4: Moment vs. lateral drift relation: (a) RC column and (b) HC-FCS column

CONCLUSIONS

The HC-FCS column exhibited high lateral drift reaching to 15.2% before the FRP rupture however, the RC-column failed at drift of 10.9%. The RC-column failed by rebar fracture and the moment capacity suddenly dropped more than 20% after that. However, the HC-FCS failed gradually with concrete compression failure, steel tube local buckling, followed by FRP rupture.

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2. *Dawood, H., ElGawady, M., and Hewes, J. (2012). "Behavior of Segmental Precast Post-Tensioned Bridge Piers under Lateral Loads", ASCE Jou. of Br. Eng., Vol. 17, No. 5, pp. 735-746.*

ACCELERATED BRIDGE CONSTRUCTION – HUEY P. LONG BRIDGE TRUSS LIFT MONITORING TO MITIGATE RISK

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ABSTRACT

The Huey P. Long Bridge is a four-span steel truss bridge that carries a two-track railroad line over the Mississippi River in New Orleans with two lanes of US 90 on each side of the central tracks owned by the New Orleans Public Belt Railroad. As part of an overall bridge widening project, and to accelerate bridge construction, pairs of 528-ft long, 2700-ton trusses were built on shore, barged in place under the bridge and connected to lifting tendons from 900-ton strand jacks and simultaneously lifted 130-ft and set in place in an 18-hour operation for the first pair of trusses. The second and third pair of trusses were lifted and set in 12 and 8 hours respectively. Since the truss would be lifted only at the end points of the 528-ft long trusses, any tilting of the truss could initiate buckling.

Therefore, the two most important parameters to measure were tilt and out-of-plane deflection of the truss. A real-time, wireless monitoring system was designed to measure truss distortions and assure overstressing or buckling of the truss did not occur during the transport, lift and setting operation. Working with the Contractor and Engineer a custom instrumentation system was developed that utilized tilt meters and laser distance sensors to control the lift. The concept of using laser distance sensors to measure deflection was developed specifically for this project. All sensors were hardwired to a data logger and continuously transmitted wirelessly to a laptop computer with multiple display panels under the bridge deck. Data were under constant review to limit ‘sweeping’ of the truss during the lifting and skidding operation.

This monitoring system helped mitigate risk to the Contractor for the three pairs of heavy truss lifts. The rationale behind the decision for making the choice of pre-building the truss sections on shore, barging and lifting into place vs. stick building trusses on the bridge will be presented with the development and implementation of the truss monitoring system.

INTRODUCTION

The Huey P. Long Bridge extends nearly 2400 feet over the Mississippi River in New Orleans, Louisiana. This cantilevered steel through-truss bridge opened to traffic in 1935 and is owned and operated by the New Orleans Public Belt Railroad. The bridge currently carries dual rail lines between the trusses and two lanes of vehicular traffic cantilevered to the exterior of each truss. The bridge carries more than 50,000 vehicles daily and approximately 30 trains per day across the only rail crossing of the Mississippi River within 100 miles of New Orleans. Based on the need to improve vehicular traffic flow and constraints due to uninterruptible rail traffic, the Louisiana Department of Transportation and Development (LA DOTD) decided to widen the bridge rather than replace it. A structural health monitoring program was included in the construction contract as a proactive measure to assess whether the anticipated amount of load would be transferred from the widening truss members to the existing truss members during the planned stickbuild construction method.

As an alternative to the stick-build truss widening specified, the MTI Joint Venture proposed a pre-built truss erection alternative to reduce impact on public, rail and river traffic requiring the lifting of three sets of paired trusses over 500-ft long and weighing more than 2700 tons. To mitigate risk to the contractor and owner, a real-time, remote monitoring system was used during the transport, lift and setting operation to limit truss distortion during the lifting and skidding operation.

Figure 1 – Original and Widened Huey P. Long Bridge Structure

The $1.2 billion widening project started with Pier widening and strengthening (Phase II) in 2006 and was completed in 2010. The Phase III of the four Phase construction included the main bridge truss widening and was awarded to the MTI Joint Venture (Massman Construction Co., Traylor Bros. Inc., and IHI Inc.) in 2007. The widening of the four spans was performed one span at a time. MTI hired HNTB to devise the span-by-span erection method. In November 2009, widening of the West Bank span began using the

stickbuild method, where each member of the span is individually placed. This is how the project was initially bid.

Figure 2 – Stick-building of Span I

In order to minimize the use of false work and river closures in the main navigation and auxiliary channels, HNTB devised a solution for the MTI JV whereby the widening of the three other spans was completed by fabricating trusses on barges near the shore and transporting the pre-fabricated trusses under the bridge and lifting them into place using strand jacks. This required the use of a 'Stability Frame'. This required two sides (upstream and downstream sides) of the pre-fabricated bridge span to be lifted and set simultaneously to avoid any unbalance forces on the bridge.

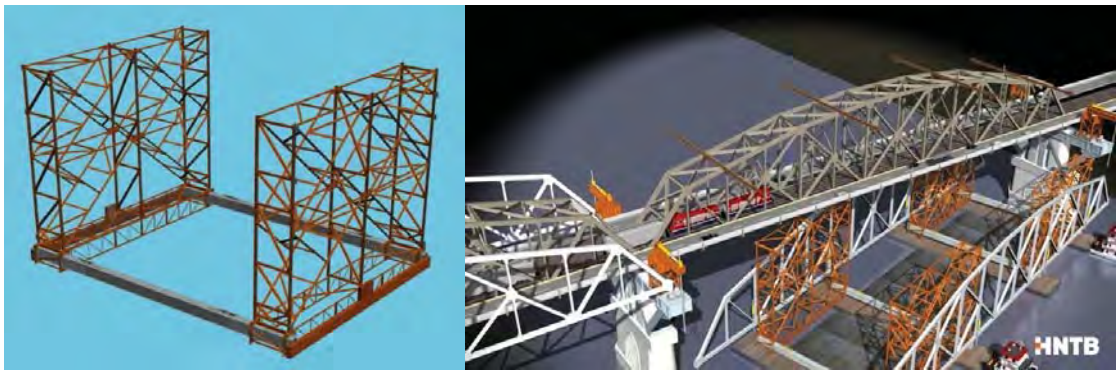

Figure 3 – Stability Frame and Positioning of Pre-build Trusses for Lift

The pre-fabricated pair of trusses in Span II through IV were assembled on barges with stability frames and floor beams near shore. MTI build a four-barge system connected by three sectional barges. HNTB designed the stability frame/floor beam assembly which was built on the barge platform to help support

the trusses and keep them plumb throughout erection. The stability frame was also used to help support the twin trusses during transport to the bridge, during the lifting operation and provided a means to skid the trusses laterally for the final setting on the piers.

Figure 4 – Truss Assembly and Stability Frame on Barges Near Shore

Construction of the first pair of twin trusses began in April, 2010 and was completed on May 31th, 2010. The first lift was performed the weekend of June 19th/20th, 2010. Subsequent lifts were performed on November 20th, 2010 and April 9th, 2011.

Table 1 – Lift Information

Lift	Span	Length, ft	Weight, tons
1	III	528	2,650
2	II	503	2,550
3	IV	528	2,700

The transport and lift operation required coordination with all four Transportation Departments (FAA- crane heights, Railroad- track closure, DOT- bridge traffic closure and Coast Guard- temporary closure of main navigational channel). The typical operation required the truss/barge assembly to be floated out to under the bridge on Friday morning and connected to lifting tendons from the 900-ton strand jacks. Early Saturday morning the bridge and navigational channel was closed and the lifting operation began.

The 130-ft lifting operation for the first lift took approximatly 12 hours with the skidding operation another 6 hours. Subsequent lifts reduced this lifting and skidding operation time down to a total of about 8 hours total for both operations. For all three lifts, the bridge was open to traffic in less than the 48-hour time requirement.

Figure 5 – Lift of Span IV. Note Stability Frames and Lifting Points are From Ends of Truss Only

Although Mammoet (lifting sub-contractor) monitored hydraulic pressures and overall lifting and setting displacements during the lifting and skidding operation, the truss (distortions) needed to be monitored to assure buckling or overstressing of the truss did not occur.

The four critical phases for monitoring included; 1) During the launching and transport of the trusses on the barges to the bridge, 2) Throughout the lifting operation, 3) During the 'skidding operation' where the trusses were pulled 13-ft laterally inwards to the bridge and set in place, and 4) The lowering of the stability frames back onto the barges.

THE MONITORING SYSTEM

The biggest concern for MTI and HNTB was out-of-plane distortion, primarily 'sweeping' of the truss. Since the truss would be lifted at the end points of the 500-ft plus long truss, any tilting of the truss could initiate buckling. Also, during the skidding operation where the trusses are pulled inward, it is important

to verify the jacking operation provides uniform movement of the truss to prevent overstressing of truss members.

Therefore, the two most important parameters to measure were tilt and out-of-plane deflection of the truss and an instrumentation layout was designed that utilized tilt meters and laser distance sensors as shown below.

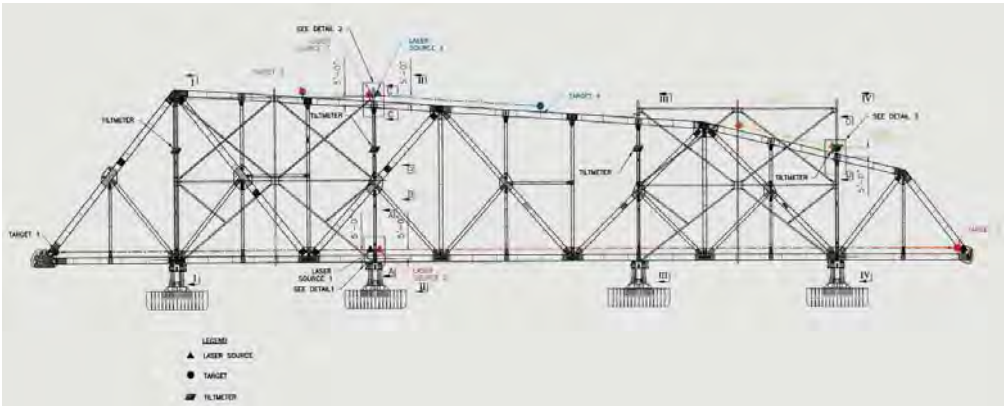

Figure 6 – Truss Instrumentation Layout

The tilt meters were located on the vertical stability frame members and measured tilt in both the longitudinal and transverse directions of the truss. Since the truss was fixed to the stability frame during transport and lifting, measurement of the vertical member of the stability frame was representative of the truss tilt. A total of eight (four on each truss/frame) biaxial tilt meters were used.

Figure 7 – Tilt Meter and Distance Lasers Fixed to Stability Frame

Laser distance sensors were used to measure truss out-of plane distortion. The concept of using distance sensors to measure deflection was developed specifically for this project. The principle is based upon the assumption that the distance does not change between the fixed laser and the fixed target, only deflection occurs. The target is mounted at 45 degrees from the deflection measurement of interest. As the truss deflects, the laser reflects at a different location (closer or further) on the target and records a difference in distance. Since the target is placed at 45 degrees, the distance change is equal to the deflection change. This concept was verified through demonstration prior to implementation.

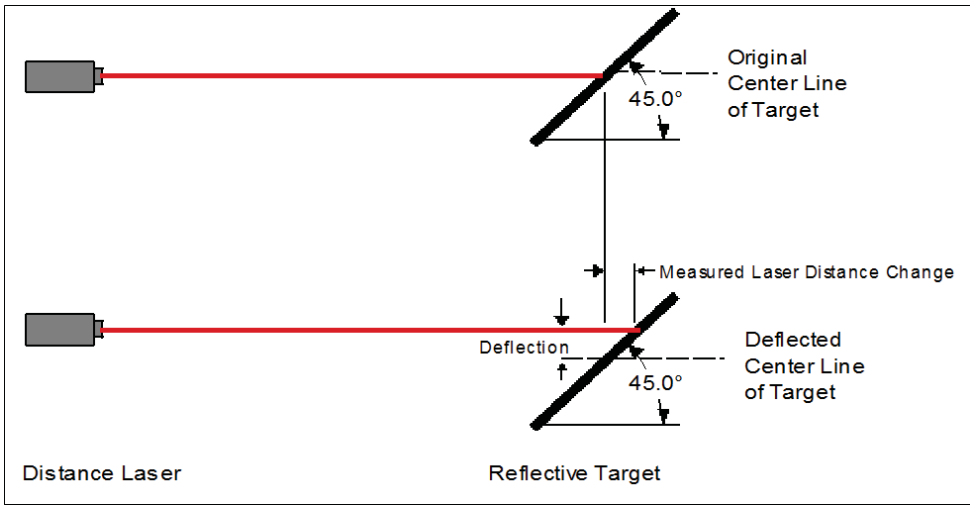

Figure 8 – Schematic Showing Concept for Distance Lasers to Measure Deflection

Figure 9 – Mounting of 45 Degree Laser Target at End of Truss to Measure Truss Deflection

A total of ten lasers were used (five on each truss) to measure out-of-plane truss distortion. The overall bottom chord end deflections were measured and three top chord deflections were measured on each truss.

All sensors were hardwired to a data logger placed on the stability frame/truss. The data logger used a 900MHz spread spectrum radio to transmit collected data wirelessly to a laptop computer with multiple displays at a remote location on top of the pier. Since the truss and data logger system would be remotely positioned on the barge during transport, lifting and setting, up to 16 hours of power was required for data collection and transmission. Data loggers were initially powered by generators to allow for maximum battery time in case there were delays in the lifting operation. Once the generators ran out of gas during the lifting operation, the system ran on battery power.

Figure 10- Data Loggers and Hardwired Sensors Transmitting Data Wirelessly to Pier Top

The monitoring station was setup under the bridge deck on top of the Pier. From this location (white box in Figure 10), communication could be maintained throughout the barging and lifting operation. Data were continuously transmitted and updated approximately every second. Data were under constant review by HNTB Engineers and decisions made for controlling lift were based on real-time truss tilt/deflection measurements.

TRUSS MONITORING

To aid in the review of data a custom interface was developed for a graphical representation of truss tilt and deformation. For each truss a ‘birds eye’ (plan) view of the truss was displayed on separate screens to show ‘sweeping’ of the truss during the lifting and skidding operation. In addition, local wind speed was monitored to prevent lifting operation during any wind speed in excess of 25 MPH.

Figure 11 – Real-time Display of Truss Monitoring System Data

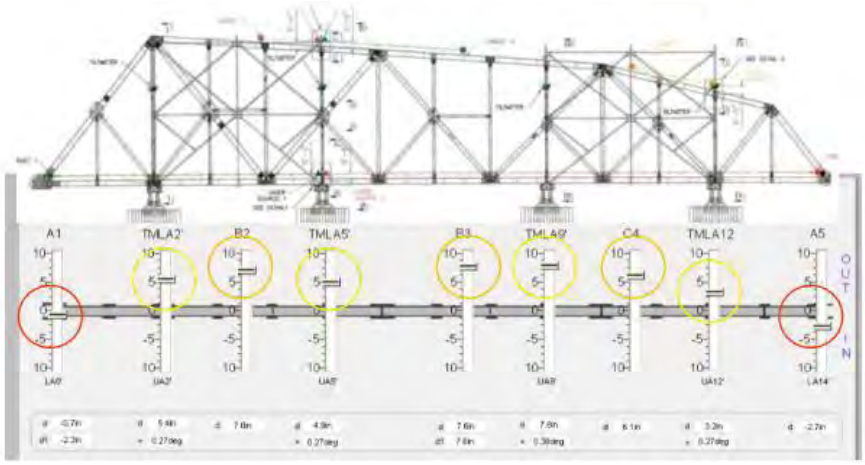

Figure 12 – Twin Screen Display Showing Plan View of Truss Distortion and Tilt

The horizontal bar on the display indicates the plan view of the top chord. The slider bars indicate the in/out deflection of the top chord at that location on the truss. For the bottom chord ends, this value is simply the deflection measured by the lasers. For the locations on the top chord, the deflection is the combination of the tilt angle x the height + laser deflection.

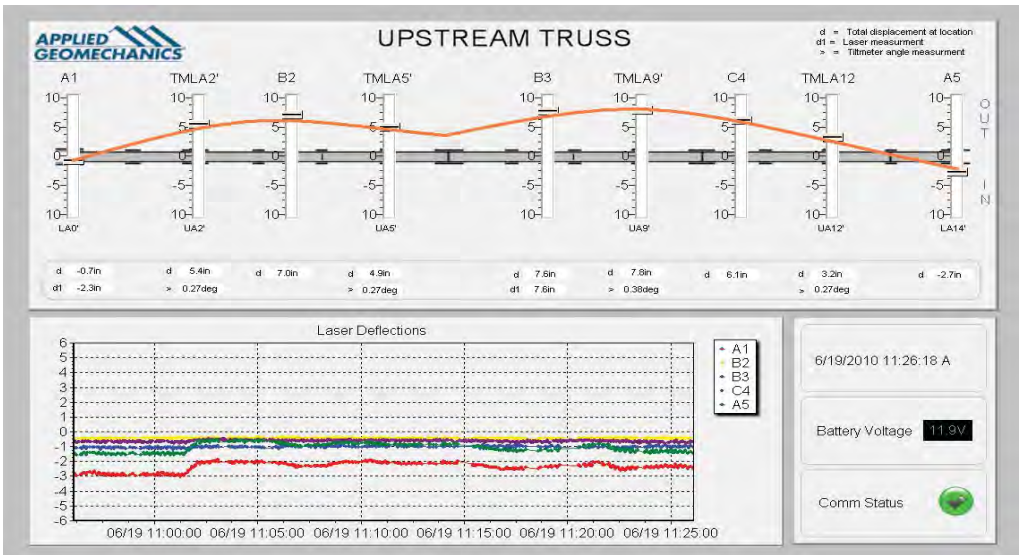

Figure 13 – Single Screen Display for Upstream Truss with Truss Distortion Superimposed

Figure 13 shows the graphical representation for the combination of truss tilt and out-of-plane deflection from tilt and laser measurements. The slider bars on the graph show displacements in inches at points along the length of the truss. The orange line is superimposed on the figure above to highlight the approximate deflected shape of the top chord of the truss as seen from a 'birds-eye' view. The bottom graph shows the time history of the laser deflections. Throughout the 8 to 18-hour lifting and setting operation, radio transmission of data to the remote monitoring station was never lost- providing constant information for the decision making process throughout the operation.

In addition to providing the Contractor and Owner a means for mitigating Risk through this barging, lifting, skidding and setting operation, the monitoring system proved its value under these specific circumstances;

- During the float-in operation for one of the lifts, the edge of one of the truss corners was hit by a tug boat during the rotation of the four-barge configuration. The plumbness and distortion of the stability frame and trusses could be immediately assessed.
- Although the lifting Contractor used crude measurements (tape measure strings) to control lifting operation, several times the lifting was stopped and adjusted based on the tilt measurements to prevent uneven lifting in both the longitudinal and transvers directions, and to prevent racking of the stability frame and truss assembly.
- During the skidding operation, the top two strand jacks had to be moved inward in-synch with the bottom four hydraulic jacks to keep the truss plumb and prevent out-of-plane bending. In one

instance (shown in figure 13), bending of the truss was noted and skidding was adjusted to allow for bottom jacks to catch up with top strand jack horizontal jacking.

- Controlled operations from monitoring system helped reduce whole operation from 18 hours to 12 hours to 8 hours for the final lift.

CONCLUSION

"The monitoring system was vital to the lift operation. We were able to use it in real time and know exactly what was happening with the lift. It allowed us to make adjustments to the attitude of the truss "on the fly" without slowing down the operation. This could not be achieved with traditional survey methods. Once we confirmed that the monitoring system was giving us results that agreed with survey and visual inspection it gave us the confidence to know how the truss was behaving at any given time. The system also allowed us to monitor the truss while it was sliding laterally into position over the bearings, which was as critical if not more critical to monitor than the lift itself." – John Brestin, formerly- Vice President and Bridge Group Director, HNTB.

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HNTB – For their use of bridge and erection renderings as well as the conclusion for this paper.

G-20: SHRP2 R04 RELATED PROJECTS

IMPLEMENTING THE SHRP2 RO4 RESEARCH RESULTS

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ABSTRACT

The objective of the SHRP2 R04 project was to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals". The aim was to develop pre-engineered standards for modular bridge substructure and superstructure systems that can be installed with minimal traffic disruptions in renewal applications. This project has developed ABC design standards, design specifications and design examples for use by engineers in the US who may be new to ABC. Two demonstration projects were also completed. The first ABC demonstration project using modular systems was completed in Iowa in 2011. The second ABC demonstration project took place in New York using a lateral slide method to replace two Interstate bridges with only overnight closures. It was completed in 2013. This paper discusses the development of the SHRP2 ABC Toolkit and the construction of the two ABC demonstration projects.

INTRODUCTION

Bridge deterioration and the need for replacement continue to be an ongoing problem in the United States. Accelerated Bridge Construction (ABC) techniques have the potential to minimize traffic dis- ruptions during bridge renewals, promote traffic and worker safety, and also improve the overall quality and durability of bridges. ABC entails prefabricating as much of the bridge components as feasible. Minimizing road closures and traffic disruptions is a key objective of ABC. For ABC systems to be viable and see greater acceptance the savings in construction time should be clearly demonstrated. The successful use of prefabricated elements to accelerate construction requires a careful evaluation of the requirements for the bridge, site constraints and an unbiased review of the total costs and benefits.

ABC applications in the U.S. have developed two different approaches; accelerated construction of bridges in place using prefabricated systems and the use of bridge movement technology and equipment to move completed bridges from an off-alignment location into the final position. Rapid construction of bridges in place offers the promise of limited closures, maybe days or weeks at the most, to allow for the complete construction of a bridge. This type of construction traditionally relies on extensive prefabrication of bridge elements including substructure and superstructure components and the use of

cranes to install these elements in their final location. As an alternative to rapid construction in-place is the use of preassembled bridges, completed at an off-alignment location and then moved via various methods into the final location using techniques such as lateral sliding, rolling and skidding; incremental launching; and movement and placement using SPMTs (Self Propelled Modular Transporters).

A key objective of the SHRP2 R04 project was to identify impediments and obstacles to greater use of ABC and seek solutions to overcome them. Focus group meetings and owner surveys identified several factors that have contributed to the slow adoption of ABC in the US. Despite the gradual lowering of costs and life-cycle cost savings, DOTs are hesitant about using ABC techniques because of their higher initial costs. Another great impediment to rapid construction is the slow engineering process of custom engineering every solution. Rather than custom engineering every solution, pre-engineered modular systems configured for conventional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors and designers.

Project R04 was composed of four distinct phases over a time period from 2007 to 2013. Phase I collected extensive data on ABC projects and identified impediments and challenges to greater use of ABC by bridge owners. The work in Phase II was to incrementally winnow down the collected findings and ABC concepts from Phase I through screening and further evaluations. The process consisted of an engineering and constructability evaluation of the concepts as well as an identification of obstacles to implementation. Phase II proposed a short list of concepts that can be advanced to design standards and the implementation phase. From this shortlist, standard concept plans have been developed for the most useful technologies that can be deployed on a large scale in bridge replacement applications. These standard concepts plans, along with design examples and recommended specifications, have been organized into a single document designed for bridge engineers; this document is the SHRP 2 ABC Toolkit, published by TRB in 2013. In Phases II and IV demonstration ABC projects were designed and constructed. In Phase IV the ABC Toolkit was extended to cover the lateral slide method of bridge replacement.

The ABC Toolkit for designers was developed to foster greater use of ABC in bridge renewal and widening projects. The ABC Toolkit is comprised of ABC design standards for the substructure and superstructure systems, design examples and sample specifications. Standardizing ABC systems will bring about greater familiarity about ABC technologies and concepts and also foster more widespread use of ABC. Using these standardized designs will serve as a training tool to increase familiarity about ABC among engineers.

As noted, an objective of the SHRP2 R04 project was to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals". The aim therefore was to develop pre-engineered standards for modular bridge substructure and superstructure systems that can be installed with minimal traffic disruptions in renewal applications. This project took the approach that for ABC to be successful, ABC designs should allow maximum opportunities for the general contractor to do his own prefabrication as there is reluctance among contractors to outsource much of their work to precasters. In this regard the R04 Team has focused on specific strategies for developing the ABC solutions under this project; the strategies can be simply summarized as follows:

- As light as possible
- As simple as possible
- As simple-to-erect as possible

Expanding on this concept, the design of components should be designed to allow the contractor to self-perform the precasting by paying special consideration to:

- Components that are simple enough to fabricate
- Components that allow some tolerance for erection
- Maximum repetition of components to reduce formwork cost
- Components that can be erected with conventional equipment like cranes

Ideally the aim would be to accomplish renewal of existing infrastructure with absolutely no interruptions to traffic or existing services. However this cannot be realistically achieved in many instances. Hence the Owner and/or Contractor should identify feasible systems that reflect a balance of cost and schedule to meet the specific objectives of every project.

To get maximum advantage from the on-site construction speed possible with prefabricated bridge installations, consideration should be given to using complete prefabricated bridge systems, including foundations and substructures. In many cases, foundation and substructure construction is the most costly and time-consuming part of constructing a bridge. This project provides design standard concepts for complete prefabricated bridge systems, including superstructure and substructure systems and foundation strategies for shallow and deep foundation systems in the context of ABC projects.

SHRP 2 ABC TOOLKIT

The use of pre-engineered standards in bridge engineering is commonplace. Many states have decided to make best use of their program dollars by greatly standardizing design through development of pre-

engineered systems, plans, etc. encompassing entire bridge systems including even the quantity take-off for various standard configurations. These are guide- line drawings that can reduce engineering calculations and details as the bulk of the calculations and details can be used for different site conditions. The use of pre-engineered bridge systems, can lead to low in-place constructed costs and improved quality. A transition of the pre-engineered but stick-built systems to pre-engineered and prefabricated ABC systems is a worthy objective to mainstream ABC and thus it was the focus of this project and specifically of the SHRP 2 ABC Toolkit.

The ABC Tool Kit was first developed for prefabricated elements and modular systems and then extended to cover the lateral slide methodology. The section of the ABC Toolkit focused on prefabricated elements is comprised of the following:

1. ABC design standards & ABC erection systems
2. Detailed ABC design examples
3. Recommended LRFD design specifications
4. Recommended LRFD construction specifications

While the SHRP 2 ABC Toolkit provides design and construction concepts for building complete bridges, this toolkit is not meant to be a complete manual on ABC, but rather, an additional resource that complements the body of knowledge and other publications on the subject. It is focused on the design and assembly of routine bridges using ABC techniques that would be of value to engineers, owners and contractors new to ABC.

Standardized designs geared for erection using conventional crane based erection will allow repetitive use of modular superstructure systems, which will make contractors more willing to invest in equipment based on certain methods of erection to speed assembly. Repetitive use will allow contractors to amortize equipment costs over several projects, which is an important component to bring overall costs in line with cast-in-place construction. Where site condition make crane based erection difficult, overhead erection using ABC construction technologies provide an attractive alternative. Both of these erection options are addressed in the ABC standard concepts in the toolkit.

A single set of ABC designs for national use in the U.S. would not be practical as there are state specific modifications to LRFD Bridge Design Specifications, including loads, design permit vehicle for Strength II and performance criteria for service limit states. For this reason the, ABC toolkit includes a set of

standard concepts for design and construction. The Engineer of Record (EOR), guided by the standard concepts and details and the ac- companying set of ABC sample design calculations will be able to easily complete an ABC design for a routine bridge replacement project. The standard concepts will need to be customized by the EOR to fit the specific site in terms of the bridge geometry, span configuration, member sizes and foundations. The overall configurations of the modules, their assembly, and connection details, tolerances, and finishing would remain unchanged from site to site. Ultimately, the ABC designs should also be reviewed for compliance with state specific LRFD design criteria.

The ABC toolkit includes typical ABC details for superstructure and substructure systems for routine bridges that are suitable for a range of spans. The standard concepts in the ABC Toolkit provide substantially complete details pertaining to the ABC aspects of the project. Much of the remaining work in preparing design plans is not particularly ABC related but more bridge and site specific customization. Specific instructions to designers are covered through general information sheets, plan notes and instructions so that all the key design and construction issues in ABC projects are adequately ad- dressed. The standard concepts, used in conjunction with the ABC sample design calculations and design specifications, will provide the transition that de- signers are looking for until they become knowledgeable about ABC.

The standard concepts included in the ABC Toolkit include the following complete prefabricated elements for modular bridges and construction technologies:

Precast Modular Abutment Systems

- Integral abutments
- Semi-integral abutments
- Precast approach slabs

Precast Complete Pier Systems

- Conventional pier bents
- Straddle pier bents

Modular Superstructure Systems

- Decked steel stringer system
- Concrete deck bulb tees
- Concrete deck double tees

ABC Bridge Erection Systems

- Erection using cranes
- Above deck driven carriers
- Launched temporary truss bridge

The design considerations for the ABC standard concepts for modular systems developed in this project are as follows:

- Substructure modules that have dimensions and weights suitable for highway transportation and erection using conventional equipment.
- Substructure modules that can accommodate deep or shallow foundations based on site requirements.
- ABC designs and specifications that allow the contractor to self-perform the prefabrication of non-prestressed components.
- Prefabricated modules designed to be quickly assembled in the field with full moment connections. Joint details that allow rapid assembly in the field.
- Modules that can be used in simple spans and in continuous spans (simple for DL and continuous for LL). Details to eliminate deck joints at piers and abutments.
- Systems that are able to accommodate moderate skews.

Typical designs for superstructure and substructure modules have been grouped into the following span ranges:

- 40 ft $\leq$ Span $\leq$ 70 ft
- 70 ft $\leq$ Span $\leq$ 100 ft
- 100 ft $\leq$ Span $\leq$ 130 ft

Connections of the modular units are important elements for accelerated bridge construction, as they determine how easily the elements can be assembled and connected together to form the bridge system. Often the time to develop a structural connection is a function of cure times for the closure pour. The number of joints and the type of joint detail is crucial to both the speed of construction and to the overall durability and long-term maintenance of the final structure. The use of cast-in-place closure joints should be kept to a minimum for accelerated construction methods due to placement, finishing and curing time. Although the ABC Toolkit focuses on more innovative materials such as Ultra High Performance Concrete (UHPC) to ensure adequate connections, post-tensioning is another recognized alternative that is

well established for ABC and for which designers can find information on from other sources. Details for substructure connections with grouted spice couplers are also provided.

The development of detailed sample design calculations for use by future designers provides a step by step guidance on the overall structural design of the prefabricated bridge elements and systems. The sample design calculations pertain to the same standard bridge configurations for steel and concrete used in the ABC Toolkit standard concepts. The intent was to provide sample design calculations that could be used in conjunction with the ABC standard concepts in the Toolkit so that the practitioner new to ABC will get a comprehensive look at how ABC designs are carried out and translated into design drawings and details.

LRFD Bridge Design Specifications do not explicitly deal with the unique aspects of large scale prefabrication including issues such as element interconnection, system strength, and behavior of rapid deployment systems during construction. The scope of this SHRP 2 project also included the identification of any shortcomings in the current LRFD Bridge Design Specifications that may be limiting their use for ABC designs and making recommendations for addressing these limitations. Recommended LRFD specifications for ABC bridge design are also included in the Toolkit. These recommendations have not yet been adopted by AASHTO.

Recommended LRFD construction specifications for prefabricated elements and modular systems include best practices that were compiled by the Research Team with the intent that they would be used in conjunction with the standard concepts for steel and concrete modular systems. As such, these specifications for rapid replacement focus heavily on means and methods requirements for rapid construction using prefabricated modular systems.

ADDENDUM TO SHRP2 ABC TOOLKIT – THE LATERAL SLIDE

In 2013 an addendum was issued to the ABC Toolkit to cover the lateral slide concept. The lateral slide is an ABC technique in which a new bridge superstructure is constructed and then moved laterally into its final position. The Lateral Slide is an option typically used for the replacement of a bridge structure on a heavily trafficked roadway and/or primary roadway where extended closures are not practical and in some cases simply not possible.

Success of the Lateral Slide is dependent upon the effectiveness of the total slide package. Coordination of all components from the conceptual design, through final design and field implementation is critical to achieving proper bridge position. The following are key components of the Lateral Slide that are covered in detail in the SHRP2 ABC Toolkit:

1. Permanent Bridge Design
2. Temporary Support System
3. Push / Pull System
4. Sliding Bearings
5. Sliding Forces

The Toolkit provides sample Drawings and Construction Specifications for the horizontal slide system and temporary shoring.

SHRP 2 ABC DEMONSTRATION PROJECTS

First Demonstration Project: US-6 Bridge Over Keg Creek, Iowa

The SHRP 2 R04 project also included field activities to pilot the concepts developed for the ABC toolkit. The first pilot project was conducted in partnership with the Iowa Department of Transportation (IADOT); the pilot bridge was located in Pottawattamie County, approximately 6 miles east of Council Bluffs, Iowa. Prior to identifying this site as potential ABC pilot, IADOT had already designed the replacement bridge using conventional cast in place; this original design had estimated construction duration, and road closure, of 6 months. Working collaboratively with the IADOT, the ABC pilot bridge was redesigned by the SHRP 2 research team and it featured several of the innovative concepts that are included in the ABC Toolkit. The replacement structure is a three-span (67’-3”, 70’-0”, 67’-3”) 210’-2” x 47’-2” steel/precast modular bridge with precast substructures and precast bridge approaches.

This pilot project incorporated the following innovations drawn from the ABC Toolkit:

- A complete bridge system was designed and constructed using superstructure and substructure systems comprised of prefabricated elements. The bridge approach slab was also a precast element.
- High Performance Concrete (HPC) was used in all precast components to enhance durability.
- The bridge has a jointless superstructure. Full moment connections between all precast components will be utilized to emulate cast-in-place construction. They also provide superior durability and minimize long-term maintenance.
- Precast pier elements were connected in the field using grouted splice sleeve couplers. They are efficient, easy to construct and ideally suited for rapid construction application.

- UHPC was used for the superstructure joints (longitudinal and transverse) to provide a durable, moment-resisting joint between deck panels. The Iowa project was the first in the US to use UHPC to provide a full, moment-resisting transverse joint in the superstructure at the piers.
- Self-Consolidating Concrete was used to improve consolidation and increase the speed of construction for abutment and wingwall piles (fill pockets) and abutment to wingwall connections.
- Use of fully contained flooded backfill to achieve rapid consolidation and significantly reduce the potential for formation of voids beneath the approach pavement.

Construction of this bridge attracted seven bidders, which is indicative of the interest in rapid construction among local contractors in Iowa and their ability to self-perform the work. Though the initial construction cost is higher than the engineer's estimate for conventional construction, the reduced installation time for the prefabricated bridge will reduce the costs to highway users. Reducing the bridge closure from 6 months to two weeks also reduced traffic control costs, reduced impact to the local economy and improved safety. Bridge was opened to traffic on November 1, 2011 after a short 14 day closure for construction.

Second Demonstration Bridge: Replacement of I-84 over Dingle Ridge Road, New York

The second SHRP 2 R04 pilot project built in 2013 involved the replacement of the existing I-84 EB & WB bridges over Dingle Ridge Road in New York State. The replacement utilized the lateral slide method. The existing structures carried two travel lanes over Dingle Ridge Road. The replacement structure for this bridge has three travel lanes in each direction. I-84 is heavily traveled, with some days having a peak AADT of up to 100,000±. The existing structure was a 3-span steel girder superstructure with a total length of 135'-0. The new structure has no skew with an 80'-0" single-span NEXT beam superstructure. The abutments are semi-integral closed-type abutments, contained by precast modular walls. Conventional bridge replacement would have extended over multiple years and would have required a temporary bridge in the median estimated to cost about $1.5 Million. Use of ABC eliminated the need for a temporary bridge and limited the traffic disruption to a single night for each bridge. This bridge replacement was utilized as a pilot for lateral slide methods and concepts that were developed under the SHRP2 R04 Project. Another goal of this SHRP2 pilot project was to showcase a concrete superstructure system for ABC. NEXT double-tee P/S concrete beams were chosen for the superstructure to minimize structure depth due to critical under-clearance issues at this site, while maintaining cost

efficiency to meet the goals of accelerated bridge construction. These new concepts were incorporated into the ABC Toolkit and published in 2013 as an addendum.

For this pilot, closure of the roadway crossing lasted one night (20 hours) per bridge to allow for the rapid demolition of the existing bridge and slide-in of the new superstructure. Maintaining traffic on the existing bridge during fabrication and assembly of the new bridge off alignment minimized traffic disruption and the costs of work zone traffic control.

Work on the new bridge was commenced while the existing bridge remained in service. The approach slabs are designed as a jump slab to temporarily carry traffic after the slide-in and before the abutment backfill is completed. The slide-in bridge replacement required low foundation settlement tolerances to prevent cracking as the completed superstructure is moved on to the new abutments. Two drilled shafts were installed at each abutment location, one to each side of the existing structure. A cap beam was cast-in-place connecting the two shafts and acted as the slide surface and a platform for the bearings for the proposed structure. Abutment drilled shaft foundations and columns were constructed to the bearing elevation of the proposed cap beam. Once the columns were in place, the precast modular walls were installed along the wing walls up to the elevation of the slide surface. Concurrently with the construction of the new substructure elements, the bridge superstructure and approach slabs were constructed on temporary shoring with sliding tracks adjacent to the existing bridges. The approach slabs consist of precast and pre-stressed concrete modules that were also connected with UHPC closure pours. The NEXT beams were supported on a cross beam / back wall cast integral with the beam ends with sliding shoes. The abutments are designed to behave as semi- integral with elastomeric bearings for expansion.

Once the initial substructure work was completed and the superstructure had been constructed, traffic was diverted to a State route parallel to I-84 during the overnight closure. The existing bridge was demolished, and the existing two lanes of the existing approach roadways were raised to the new profile. A precast sleeper slab was placed at the end of either approach to serve as an end dam for the raised approaches and also to serve as a sliding surface for the approach slabs. The superstructure and approach slabs were then slid in place on PTFE pads, which offer very low friction resistance. Jacks were used to push the new superstructure on to the new abutments. As the approach slabs were designed to take live loads temporarily, traffic was re-routed onto the new structure at this point. With the new structure in place and traffic restored to the eastbound crossing, substructure and approach roadway work was finalized. The remaining precast modular wall sections were placed along the abutment wing walls, up to the approach slabs. Controlled low-strength flowable fill was used to complete the backfill under the approach slabs and create positive contact between the approach slabs and underlying subgrade, while under live traffic.

Approach pavement work was performed to raise the widened roadway to the same elevation as the raised section carrying two lanes. A final 2” asphalt course will be applied across the new approach roadways and bridge to meet the final profile. The ABC timeframe is considered to begin with the closure of the existing crossing and to end when traffic is rerouted over the new bridge. The 20-hour closure was scheduled to occur over a weekend, commencing Saturday night and extending through Sunday morning. The project was completed in October 2013. The first ABC period was scheduled in September 2013. Replacement of the West-bound Bridge was performed during the second weekend in October 2013.

CLOSING

ABC is fast becoming a very powerful tool in the transportation industry. ABC provides smarter and faster ways of rebuilding deficient bridges using standardized approaches that allows economies of scale in manufacturing and construction, and increased safety. ABC is beginning to gain momentum nationally as owners begin to consider the added value of safety, quality and the impacts of long-term road closures for construction, in determining the total cost of a project. Design standards and design examples provided in the ABC Toolkit are intended to assist designers and owners new to ABC. Applying the guidelines provided in the ABC Toolkit, designers can easily implement ABC technologies in their bridge replacement projects.

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SHRP2 INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL: LESSONS LEARNED

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ABSTRACT:

The SHRP2 project, *Innovative Bridge Designs for Rapid Renewal* (R04), is coming to a successful close and has produced several positive outcomes and products that can be used by all transportation agencies. The first is a design toolkit, which offers information on how a transportation agency might approach the process of replacing an existing bridge in a greatly accelerated manner. This process includes prefabricating most of the bridge off-site or in some cases sliding the bridge in from the side with a roadway closure of less than one day. Best practices from around the country were gathered in the toolkit and various methods were tested by eight "lead adopter" transportation agencies during 2014 and 2015. Three showcases were held by the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the lead adopter states as well as three Peer-to-Peer exchanges covering all 50 states and the District of Columbia. The showcases gave the lead adopter states an opportunity to highlight its bridge project and the many lessons learned when applying the *Innovative Bridge Designs* Toolkit. The Peer-to- Peer exchanges also offered transportation agencies at the state, Federal, and local levels an opportunity to share and learn from each other's experiences.

This paper presents a compilation of the lessons learned during the SHRP2 *Innovative Bridge Designs for Rapid Renewal* project with an emphasis on the experiences and observations of the transportation agencies as they move into the accelerated bridge construction world.

Key Words: Accelerated Bridge Construction, ABC, SHRP2, Lessons Learne

INTRODUCTION

With so many small- and medium-sized bridges needing replacement, state departments of transportation (DOTs) need innovative design options deploying precast alternatives. As part of the second Strategic Highway Research Program (SHRP2), the *Innovative Bridge Designs for Rapid Renewal* (R04) product was developed to answer that need. At the core of the product is the *Innovative Bridge Designs for Rapid Renewal* Toolkit. The toolkit includes standard design plans for foundation, substructure and superstructure systems, subsystems, and components that can be installed quickly with minimal traffic disruptions. It also provides detailed standards and design examples for complete prefabricated bridge systems (PBES), as well as flexible design concepts that can be adapted to most small- and medium-sized bridges. The toolkit, and slide-in bridge addendum, is available on-line at http://www.trb.org/Main/Blurbs/168046.aspx.

Under the SHRP2 Implementation Assistance Program, the *Innovative Bridge Designs for Rapid Renewal* Toolkit has been successfully deployed on ten state DOT projects including a variety of PBES bridge elements, construction settings, and erection techniques. In addition, three national project showcases were held beginning in fall 2014, and the first of several regional peer-to-peer exchanges were held starting in late-spring 2015. Product deployment goals include providing simple design and construction planning tools to implement PBES options and sharing the latest Accelerated Bridge Construction (ABC) techniques with an emphasis on "states talking to states." During the various stages of product deployment, numerous lessons were compiled regarding ways to improve PBES designs, project acquisition, and ABC processes. The following is a compilation of these lessons with insights from national ABC experts who presented at the Sacramento SHRP2 Peer-to-Peer Exchange in May 2015.

Overview of the Sacramento Peer-to-Peer Exchange

During the two-day Sacramento Peer-to-Peer Exchange, public transportation agencies shared ideas and best practices about PBES design options, preferred contracting approaches, construction approaches, and the numerous challenges to successfully implementing PBES in an ABC environment. Representatives from each state presented their unique experiences and answered questions from their peers. As expected, the level of ABC experience varied greatly among the states.

The second day also included the participation of consultant designers, contractors, and academia, and featured several presentations from national ABC experts as well as open discussions on the challenges of integrating ABC into the mainstream of bridge design/construction. State participants generally agreed that the issues and challenges facing the implementation of ABC across the states are more alike than

different. These observations are included in the following subsections and later in the summary of lessons learned.

State DOT Perspectives

Each attending state was asked to give a short presentation on their state's ABC activities. The presentations and follow-up discussions became the real heart of the two-day exchange. Below are the major points from the presentations (more details are included in the Lessons Learned section later on):

- ✓ DOTs are complex and you need a champion and real top-down support to create effective change.
- ✓ Work closely with all parties affected by the introduction of ABC, such as:
  - DOT design, permitting, acquisition, and construction groups
  - Consultants
  - Contractors
  - Public
  - Media
- ✓ ABC costs are complex. Look at the big picture, including such things as traffic control, construction staffing, safety issues, and the needs of the traveling public.
- ✓ Safety and quality will always be the leading drivers of change within a DOT – speed of construction, regardless of cost, drives the public.
- ✓ Having an ABC decision-making system (simple or complex) is required to clearly understand what project-specific factors drive the potential use of ABC.
- ✓ Environmental issues may dictate the need to consider ABC.
- ✓ The DOTs garner great public and political gain from successful ABC projects.

A large-group discussion among DOT participants drew particular interest in the various contracting practices and strategies for ABC projects. The traditional design-bid-build process works, but may constrain the designer and/or contractor from fully exploring the various ABC options. It was pointed out that some of the best ABC methods have come from design-build projects where the designer and contractor work hand in hand. In the design-build process, time really is money and the push for speed is real. A very productive discussion then followed on the Construction Manager/ General Contractor (CMGC) method of contracting. In this process, the owner hires a contractor early in the design process to work side-by-side with the design team. These projects have generally been very successful, providing for equitable risk-sharing and encouraging collaborative innovation to align final designs with the contractor's best means and methods. The SHRP2 Gila River project in Sacaton, AZ (Figure 1) was

designed/constructed under the CMGC method of contracting and all three parties -- owner, designer, and contractor -- were very pleased with the process and results. It was clear from all involved that the CMGC method should be added to our list of contracting methods for future use.

Figure 1. Highly successful CMGC bridge slide February 2015 at the Gila River Indian Community near Sacaton, AZ.

Based on discussions with the DOT participants, ABC does generally cost more than traditional construction. What tends to lead to these additional costs is perceived risks by the contractor about not making the ABC window. Penalties are real and contractors bid risk just like labor and materials. The need for speed needs to be carefully measured along with the inherent costs of speed. For example, weekend projects are great, but cost more than 14-day projects. Some of the more direct ABC issues or risks contractors consider are project disincentives, owner submittal review time, responsiveness of sub-contractors, need for larger or specialized equipment, and other items the contractor sees as beyond their control during a short ABC window. All of these items add to a contractors' risk and hence the ultimate bid price. That's why deciding just how fast a project "needs" to be completed is a critical step in the ABC process.

When figuring the actual owner cost of ABC, the cost of the bridge needs to be considered, but so do the potential savings in traffic control, construction management, and safety of shortening the impact to the public. User delay costs are real to the users and should be considered in some form when looking at the need for ABC. Detours cost users both time and money – not to mention the frustration they feel. Several states (Utah, Connecticut, Minnesota, Oregon, and others) have set up formal processes to evaluate the impact of ABC on bridge construction projects and determine whether an ABC approach should be considered or implemented. Although these figures can be difficult to calculate, simply calculating the basic square footage cost of the structures does not always represent the full picture. Stepping back and looking at the total project cost can be an eye opener to those watching the budgets.

Contractor Perspectives

Contractors price three main items – labor/equipment, materials and risk (e.g., schedule, site conditions, weather, environmental impact). In traditional design-bid-build they understand these well and price them accordingly.

When the project construction process is accelerated, the risk factors increase. If additional subcontractors (precasters) are added, the risk increases again as there is another step beyond the direct control of the prime contractor. It was pointed out during the Peer-to-Peer Exchange that much of this risk can be either mitigated or at least assigned to a specific entity in the CMGC process. When the owner, designer, and contractor work together in a collaborative way, risks can be defined and assigned to the party best suited to handle the risk. This may indeed be the owner, but if the project price is positively affected by this approach, the risks may be worth the potential money savings. Fitting the proposed construction process to the contractors' means and methods also reduces the risks, thereby lowering the price. The contractors in attendance liked the CMGC process and would like to see this contracting tool used more often in the future.

National Expert Perspectives

Three different speakers talked about ABC from a national perspective. One of the recurring themes was to "keep it simple." Complex solutions rarely work out well in the field and always cost more money. Trying to use simple details that allow contractors to use equipment they already own and understand leads to the best combination of quality and price. Precast elements are generally of higher quality than cast-in-place concrete and add value to the final constructed project. Care must be taken to ensure the element joints are detailed well so they will be both long lasting and fit together well when assembled in the field.

One of the ABC issues brought up was expansion deck joints. Experts noted that expansion joints have been problematic over the years so they are generally avoided. Sound deck joints between precast elements need to be part of any proposed ABC solutions in order to be at least as good as the cast-in-place option the ABC method is replacing. SHRP2's *Service Life State Design* initiative fully supports the elimination of expansion-type deck joints to the fullest extent possible.

Experts also discussed the positive impact ABC has on work zone safety. In 2010, 576 fatalities occurred in construction zones. ABC dramatically improves work zone safety by minimizing worker exposure to oncoming traffic and traveler exposure to construction zone hazards. This is a real benefit of ABC and one that is challenging to quantify, but needs to be considered.

Experts noted that Slide-In Bridge Construction (SIBC) is a nice combination of speed and cost. Self-Propelled Modular Transporters (SPMT) can be fast, but can be costly to bring on site. Assembling numerous precast bridge components piece-by-piece will save money, but may take several weeks to complete. All three of these methods have a place in the ABC world. Site conditions and the need to build fast will normally lead the project team to the best solution. Always step back and consider the bigger picture as you consider using an ABC solution.

Lessons Learned

The following "lessons learned", grouped by general category, have been compiled from DOT, contractor, and national expert participants at recent SHRP2 showcases and peer-to-peer exchanges. Some of the items may seem obvious but bear repeating since they are important. Others are less obvious and should be considered when looking at building an ABC project.

Cost of ABC

1) As with the introduction of most new technologies, some states are seeing a decline in ABC costs with similarly designed, repetitive ABC projects, and increased acceptance by contractors.
2) ABC is showing up on many design/build contracts. This should mean something to all of us. ABC can be an effective tool to assist the contractor in expediting the project and is cost effective in the bigger picture at times.
3) Owners need to step back and look at the big picture when it comes to project costs and consider:
   a) Cost of traffic control (how long in place).
   b) Cost of staff (construction management).
   c) Other items such as safety to the public are hard to quantify, but are real public benefit.

4. Standardization of ABC approaches saves money.
5. Simple details lower costs. Complex details cost more money. Think simple!
6. Precast fabrication tolerances have a real effect on project costs – keep tolerances realistic and attainable, giving due consideration to the issues impacting fit-up on the job site.
7. Using an ABC process that aligns with the local contractor's equipment can help control costs.
8. Engaging all design and construction parties involved from early on in the project through construction ensures that unforeseen issues, onerous specifications, submittal review issues, or other issues don't impact the accelerated aspects of the project.

Speed of ABC

1. In some areas, the length of the detour is a real driving factor for ABC.
2. ABC is not just about building the bridge fast, but also mitigating traffic, safety, and environmental impacts.

3. Self-Propelled Modular Transporters (SPMT) work well, but are pricey. However, if speed is the main goal, SPMT may be the right solution.
4. Slide-In Bridge Construction (SIBC) can be a cost-efficient form of fast ABC if the site is appropriate for this approach.
5. The use of precast is common in the ABC world although there are times when some elements can be cast-in-place as fast as precast elements and should be an allowed option for the contractor.

Policy-Related Issues

1. There are many ABC resources available. Check the FHWA website along with Utah and other states that have published manuals and best practices: http://www.fhwa.dot.gov/goshrp2/Solutions/Renewal/R04/Innovative_Bridge_Designs_for_Rapid_Renewal
2. Several states (Minnesota, Oregon, Utah, Connecticut, and others) have developed several different ways to decide if an ABC approach is appropriate for projects during the scoping phase.
3. Consider the real goal of the project. Is it ensuring the least disruption to the traveling public or lowest cost to the owner agency? This question needs to be asked and answered early in the process.
4. States need to look at opportunities to allow the prime contractor to cast elements without the precast certification requirement. This would make it more attractive to bid ABC projects since they would retain more of the work. (This concept does not include precasting prestressed elements.)
5. ABC is not the right approach for every project. The specifications of the project and the local conditions need to be considered carefully when exploring an ABC approach.
6. Several states now have a chapter in their Bridge Manual devoted to ABC. At this time, 44 states consider ABC during project development with 6 states running dedicated ABC programs.
7. Scope projects according to the goals for the project, not just cost.
8. Environmental issues can be a strong driver for ABC. For example, an ABC project was built in upper Michigan in the middle of the winter with precast elements. The main driver was the sensitive marsh land in a wildlife preserve. With the marsh frozen, minimal effects were seen from the construction activities and the project was completed before the spring thaw.
9. Length of construction season can be a strong driver for ABC. Projects that in the past took more than one season can now be competed in one season, preventing the construction site from being exposed to harsh winter conditions.

Owner-Related Issues

1. DOTs need to innovate if they want to meet user demands related to structure capacity, availability, and durability.
2. Do not be afraid to try something new as long as you exercise good engineering judgment supported with proven technology (such as the SHRP2 PBES toolkit).
3. Durable connections between precast elements are a must to gain acceptance with owners.
4. Changing the "we have always done it this way" paradigm is not easy. You need top-down support and a team approach to effectively change the current environment to promote ABC.
5. DOTs can gain tremendous political capital by employing ABC on their projects (Figure 2).
6. There were 576 fatalities in work zones in 2010. ABC can improve work zone safety by cutting down the exposure of the public to the contractors operations (and vice versa).
7. Using precast elements on an ABC project improves the quality of the final bridge. Better concrete mixes and more controlled curing lead to these durability improvements.

Figure 2. Local media interviewing Wisconsin DOT ABC Project Manager about accelerated construction activities along the USH 29 corridor near Green Bay, WI.

Technical Issues

1. Geotechnical issues can be a major concern on ABC projects. Consider closely the method of structure support as it directly affects the project's progress in many cases. Spread footings are fast and inexpensive if applicable.
2. Post tensioning in an ABC world tends to take extra time.
3. Keep it simple. Don't overcomplicate designs to be successful with your ABC projects. Meeting with contractors before bid is critical when new ABC methods are being rolled out.

4. Ultra High Performance Concrete (UHPC) is a useful tool but needs to be used only when needed since it is expensive. (States are developing their own generic versions of UHPC at this time.)
5. Geosynthetic Reinforced Soil (GRS) is a fast and simple way to build abutments and/or approaches. Scour remains a concern when they are used for water crossings.
6. Pay close attention during the shop plan review process. This is important and well worth the effort in order to prevent field issues.
7. Bridge deck overlays can help with minor deck vertical alignment issues.
8. Be aware of the weight of precast elements. Having to mobilize and use big cranes can be costly.
9. Precast spread footings, when applicable, promote fast construction.
10. ABC can help solve some weather-related issues, such as cold temperatures or rain.
11. Avoiding expansion deck joints reduces future maintenance costs. Reduced joints need to be part of most ABC methods.
12. Grouted reinforcing splice couplers are well suited for ABC projects. Tests have shown they have good seismic properties. Be careful to check bar clearances in locations of bar couplers.
13. Durability of joints is not an issue if quality details are used. Keep joint details simple and, if desired, have the precast elements fitted to each other in the precast yard to guarantee a good fit in the field.

Contractor-Related Issues

1. Contractors prefer to retain as much of the work within their company and direct control as possible.
2. Working closely with the contractor before, during, and after the project is key to long-term success of ABC. A debriefing meeting after the project is complete can yield many good lessons learned to bring forward to the next ABC project.
3. Contractors bid risk along with labor and materials. Risk tends to lower with each completed ABC project.
4. The prime contractor makes money building things, so leave as much of the project in their hands as possible.
5. Contractors like CMGC contracts because these work to their strengths.

Contracting with ABC

1. CMGC contracts can work very well as long as all parties (owner, designer, and contractor) truly buy in. (Several good examples of CMGC projects were cited during the exchange.)
2. Contractors know a thing or two about building bridges. Listen closely to their suggestions for improving the ABC process.

3. Most ABC projects go well. This is often attributed to the emphasis on good communication between all the involved parties. Good communication equals good projects!
4. Owners like the direct input and better control they receive under a CMGC contract than the less controlled design/build process.
5. How tight an ABC schedule is set dictates risk and costs to the contractor. Pick your timeline carefully and consider what the project "really" needs. An extremely fast timeline can be costly.
6. CMGC contracts often come down to each party (owner, designer, contractor) being willing to accept the input of others to find a good solution. Be open to suggestions.
7. CMGC contracts are all about trust. Build trust and the rest is simple.
8. Assignment of risk during the CMGC process can help reduce the overall project costs.
9. Incentives/disincentives need to be carefully used in contracts. Too little and they are meaningless. Too much and the disincentives lead to big risks for the contractor and costs to the owner.

General ABC Topics

1. The "A" team is critical for the first several ABC projects. This applies to the owner, designer, and contractor.
2. At some time in the not-too-distant future, we will call these methods BC (Bridge Construction) and not ABC because this approach will become the norm. The times are changing.
3. Not every project needs an ABC solution, but some really benefit from the approach (Figure 3). Identify the projects that could most benefit from an ABC solution and boldly move forward!

Figure 3. Superstructure ready to be picked and swung into place on Warren Avenue project, Providence, RI.

Summary

The SHRP2 showcases and subsequent peer-to-peer exchanges have been a success. Owners had the chance to hear what the ABC-related issues were in other states and many realized they were facing similar issues and concerns. Designers had the chance to hear the issues and concerns owners are experiencing with ABC and how they would like to see them addressed on future projects. Contractors had the opportunity to share their concerns about the ABC method of construction directly with owners and designers. It's clear from the discussion that contractors not only factor in labor and materials into the price, but also the potential risk of the project.

As the ABC methods mature, all parties involved in the process will become better informed about the costs and benefits of building bridges with an accelerated approach. Prefabricated bridge elements and systems as presented in the *Innovative Bridge Designs for Rapid Renewal* Toolkit are good examples of how the bridge building industry can reduce user delays and improve quality and safety of the constructed project. These peer-to-peer exchanges have, as one attendee noted, "Lit the fuse of the ABC world" so we can all see the benefits of this growing methodology.

The authors would like to thank the presenters at the Sacramento Peer-to-Peer Exchange for their thoughts and observations on the current ABC world. In particular thanks to Dorie Mellon, Mark Case, Mike Culmo, James Luebke, Bill McEleney, Farshad Mazloom and the state representatives for sharing their thoughts and experiences with the group. This made for a very successful ABC Peer-to-Peer Exchange.

SACATON ROAD OVER THE GILA RIVER – AN ACCELERATED BRIDGE REPLACEMENT PROJECT

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EXTENDED ABSTRACT

Sacaton Road (BIA/IR Route 7) is a 2-lane, north-south road that serves as a primary commuter and access route for the residents of Sacaton, AZ, the capital of the Gila River Indian Community. The current Average Annual Daily Traffic on this road is over 5000 vehicles per day. This traffic is forecast to grow

to over 7500 vehicles per day by the year 2034.

Immediately north of town, Sacaton Road crosses the Gila River, a typical Arizona desert river that can go from bone dry to a raging torrent of water, with little warning. The river has incised a relatively narrow thalweg that is approximately 140 to 150-feet wide along one edge of the nearly 1-mile wide floodplain.

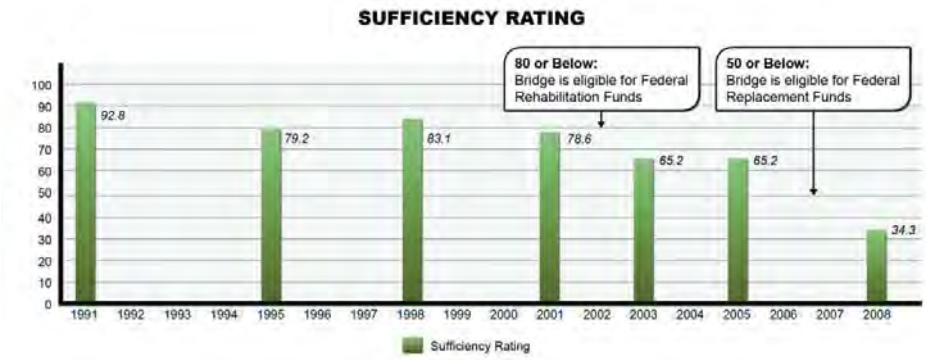
The existing road is carried over this thalweg by a 4-span, precast rectangular beam bridge constructed in 1961 by the Bureau of Indian Affairs. The remainder of the floodplain is crossed with an at-grade

roadway that is armored along its downstream edge to protect it during larger flow events that periodically overtop the road, shutting down access to and from Sacaton.

The existing bridge was supported by skewed, solid concrete pier walls and stub abutments founded on relatively shallow driven steel H-piling. The substructure was skewed 45 degrees right to the roadway centerline. The bridge carried two 12-foot wide lanes with 2-foot wide shoulders on each side.

Based on the last bridge inspection performed in 2008, the existing Gila River Bridge was rated in poor condition and its barriers/approaches did not meet current standards. In addition, the existing bridge width did not meet current AASHTO standards.

The existing bridge was beyond its useful design-life. Moreover, the most recent bridge inspection information reveals that the condition of the bridge has steadily deteriorated over the past 20 years, and was beginning to accelerate to the extent that the bridge deck was rated in Serious Condition (rating is 3 out of 10), the superstructure was rated in Satisfactory Condition (6), and the substructure wass rated in Poor Condition (4). The latest bridge inspection assigned the bridge a Sufficiency Rating of 34.3 (out of 100), making it eligible for Federal Bridge Replacement Funds.



In addition to the deteriorating structure, the existing bridge configuration with skewed pier walls was hydraulically inefficient because the flows approach the bridge nearly normal to the roadway. This configuration creates backwater effects that cause smaller events to overtop the roadway north of the bridge.

Faced with the need to improve the roadway safety, replace the bridge and decrease the frequency of roadway overtopping, the Gila River Indian Community Department of Transportation (GRIC-DOT), in partnership with the Federal Highway Administration's Office of Federal Lands Highway and the Tribal

Transportation Program, set out to conventionally design, bid and build a typical bridge replacement project. The culmination of this effort produced a set of "typical" Bid Ready Documents for a conventional bridge replacement, but also a couple unresolved challenges:

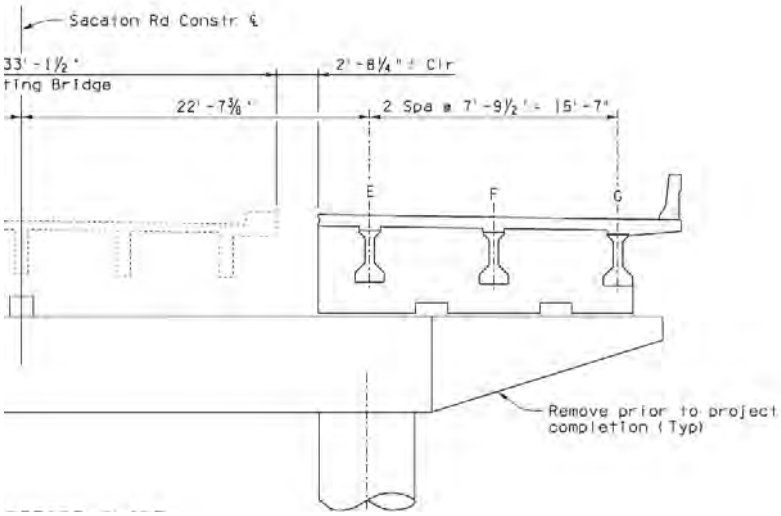
1. A relatively long duration of many months of direct traffic impact resulting from a full closure and 6 mile detour of Sacaton Road, while vetted and "accepted" during the design development, was still unsettling because of the negative affect it would have on the residents of Sacaton.

2. A Cost Estimate of $2.7 million with a budget of only $2 million.

An opportunity presented itself to solve both of these challenges when the GRIC-DOT submitted the project for and was awarded a Strategic Highway Research Project 2 (SHRP 2) Grant that infused the project with $500,000 to implement components of the recently released SHRP2 ABC Toolkit – Innovative Bridge Designs for Rapid Renewal. In addition, the project partners sought to achieve the most value from this opportunity and swapped the conventional Design-Bid-Build approach for the successful Construction Manger General Contractor (CMGC) delivery method.

This presentation will explore a selection of the challenging boundary conditions, problems and solutions that the team encountered while redesigning the project into an FHWA Demonstration Project for the

SHRP 2 ABC Toolkit and the technique chosen to accelerate the construction of the bridge and reduce the months long impact on travelers to just days. The presentation will include tangible results, experience and highlights directly from the construction of the bridge, along with lessons-learned from the chosen techniques used to build the new bridge in halves on each side of the existing bridge and slide these elements into their final configuration, scheduled for early 2015.


Sacaton Rd Constr ℄
33'-1½"
ting Bridge
2'-8¼" ± Clr
22'-7⅜"
2 Spa @ 7'-9½" = 15'-7"
E
F
G
Remove prior to project
completion (Typ)