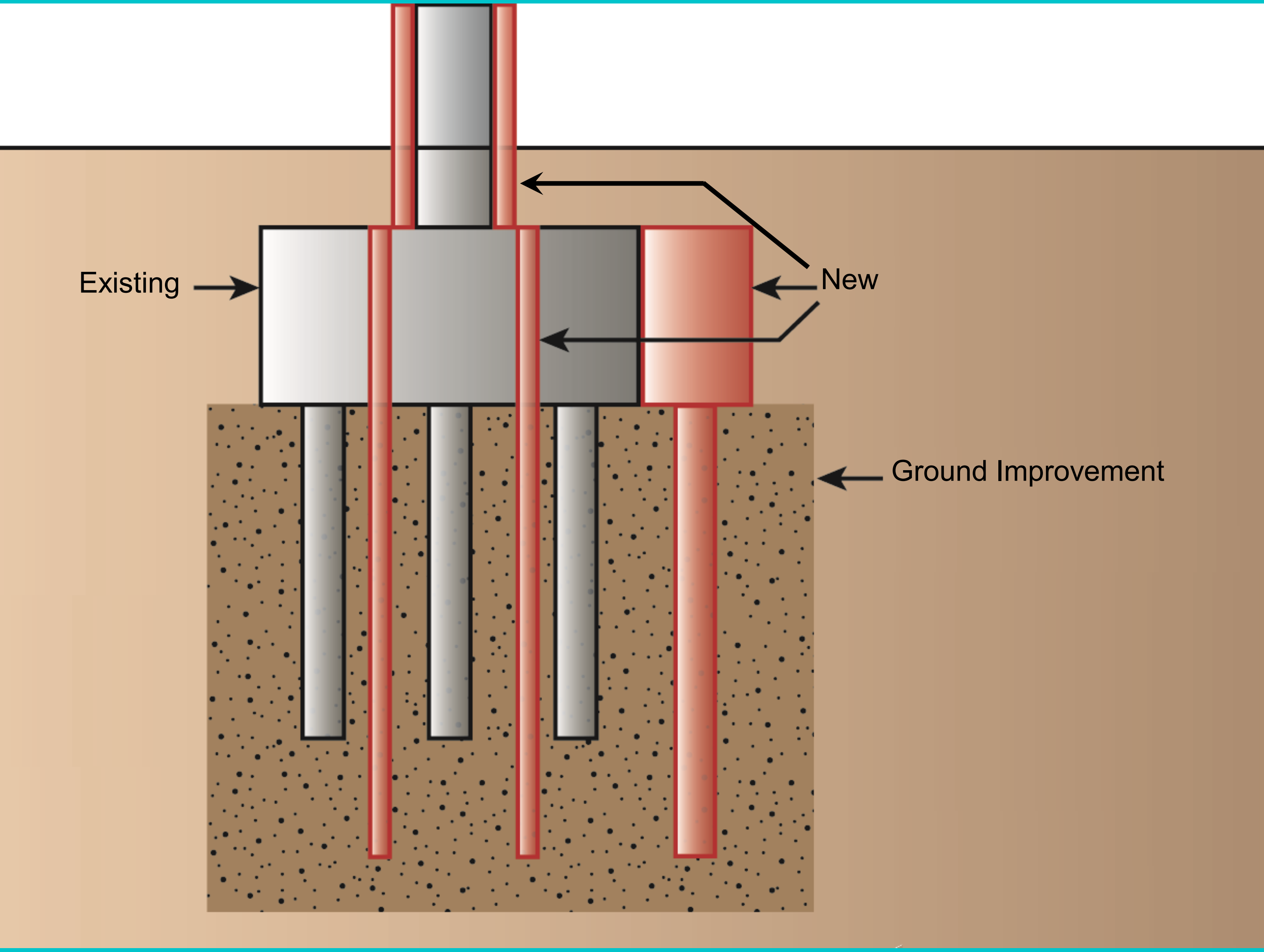
Publication No. FHWA-HIF-18-055
Infrastructure Office of Bridges and Structures
November 2018

Foundation Reuse for Highway Bridges



U.S. Department of Transportation
Federal Highway Administration

Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

FOREWORD

Given the high percentage of deteriorated or obsolete bridges in the national bridge inventory, the reuse of bridge foundations may be a viable option that can present a significant cost savings in bridge replacement and rehabilitation efforts. The potential time savings associated with foundation reuse can, in turn, reduce mobility impacts and increase the economic viability and sustainability of a project. However, existing foundations may have uncertain material properties, geometry, or details that impact the risks associated with reuse. Unlike a new foundation, an existing foundation may have been damaged, may not have sufficient capacity, and may have limited remaining service life due to deterioration.

Assessment of these issues as well as foundation strengthening and repair measures and innovative approaches to optimize loading are discussed in this report. To better demonstrate the engineering assessment of key integrity, durability and load carrying capacity issues, the report contains fifteen (15) case examples where foundation was reused by the owner agencies. On new construction, the report looks ahead and includes discussions on foundation design with consideration for reuse.

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Research and Development

Report No. FHWA-HIF-18-055	2. Government Accession No.:	3. Recipient's Catalog No.

4. Title and Subtitle:
Foundation Reuse for Highway Bridges

5. Report Date:
November 2018

6. Performing Organization Code:

7. Authors:
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8. Performing Organization Report No.:

9. Performing Organization Name and Address:
The City College of New York
160 Convent Ave, New York, NY 10031

10. Work Unit No. (TRAIS):

11. Contract or Grant No.:
DTFH61-14-D-00010-T-14001

12. Sponsoring Agency Name and Address:
Office of Research, Development, and Technology
Federal Highway Administration
6300 Georgetown Pike
McLean, VA 22101-2296

13. Type of Report and Period Covered:

14. Sponsoring Agency Code:

15. Supplementary Notes:

16. Abstract:

Foundations of existing highways and over-river bridges may have significant functional value. Hence, reuse of foundations of existing bridges during reconstruction or major rehabilitation can result in significant savings in costs and time. This report on bridge foundation reuse addresses critical issues encountered during decision-making on foundation reuse, assessment of existing bridge foundations for integrity, durability and capacity, strengthening of bridge foundations / substructures and design of bridge foundations for future reuse. The report includes numerous case examples on reuse of bridge foundations in the U.S. and Canada to highlight significant benefits of foundation reuse from social, environmental and economic perspectives. These case examples also present a detailed process followed in resolving integrity, durability and capacity issues encountered during the reuse process, and will serve as a knowledgebase for transportation agencies interested in reusing bridge foundations. Planning for reuse during the construction of a new bridge is a very important sustainability initiative that has also been addressed in this manual. This document is not meant to be used as a guideline; only as decision-making tool in addressing technical challenges and risk in reusing bridge foundations.

17. Key Words:
Foundation reuse, substructure reuse, accelerated bridge reconstruction, integrity assessment, durability assessment, capacity assessment, substructure strengthening

18. Distribution Statement:
No restrictions. This document is available to the public.

19. Security Classification (of this report): Unclassified	**20. Security Classification (of this page):** Unclassified	**21. No of Pages:** 234	**22. Price:**

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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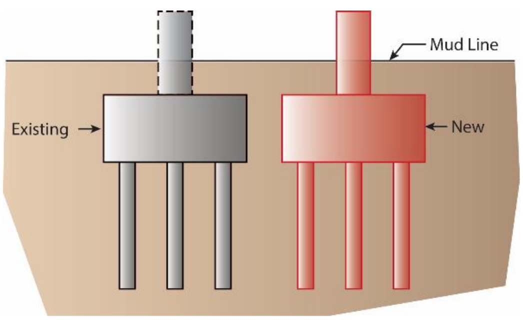
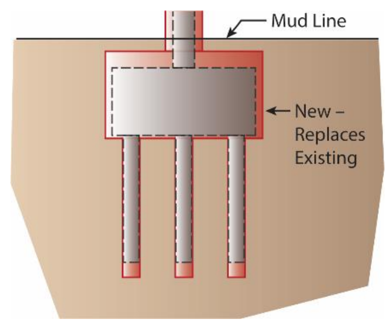
CHAPTER 1. INTRODUCTION TO FOUNDATION REUSE

DEFINITION OF FOUNDATION REUSE

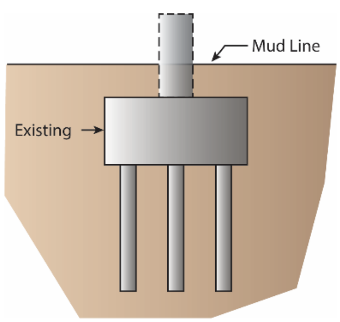
To date, “foundation reuse” has not been defined clearly in technical specifications or design codes. Based on a detailed literature review and discussion with various stakeholders, foundation reuse in this report is defined as ***use of an existing foundation or substructure of a bridge, in whole or in part, when the existing foundation has been evaluated for new loads***. Foundation reuse includes the reuse of substructure components both above and below ground, including rehabilitation of existing substructure and foundation elements when the superstructure has been replaced. Reuse of the existing foundation when superstructure loading has been optimized by using innovative solutions such as lightweight concrete, fiber reinforced plastic, etc., is also considered as foundation reuse. Other situations where foundation reuse may occur include bridge superstructure replacement, bridge widening and repurposing, and retrofitting for seismic or other purposes. In-kind bridge deck replacement, where there is no change in the loading, is not considered foundation reuse in this manual unless a new evaluation of the foundation loading and/or capacity was undertaken. Figure 1 illustrates four different options (Jalinoos 2015) available when replacing an existing bridge foundation.



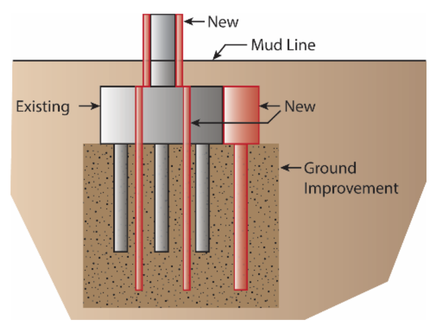
A. Option 1: Install new foundation on new alignment



B. Option 2: Install new foundation on the existing alignment



C. Option 3: Reevaluation and reuse existing foundation



D. Option 4: Reuse existing foundation by strengthening it

Source: FHWA

Figure 1. Illustration. Foundation reconstruction options

Option 1 involves the construction of a new foundation on a new alignment while avoiding the existing foundation. Construction of the new elements does not interfere with the existing foundation or impact user mobility (although there may be mobility impacts while switching to the new alignment). In Option 2, the existing alignment is maintained, although on new substructure elements. The new substructure elements may be constructed with the original bridge in service, or after closure to traffic. This option may require demolition of existing substructure elements prior to reconstruction. In Option 3, the existing foundation is reused as is, with or without minor repairs such as patching or chloride removal. In Option 4, foundations are reused with some form of retrofitting or strengthening. Both Options 3 and 4 illustrate the case of *foundation reuse*. These options may contain substructure elements not depicted in figure 1 that may or may not be suitable for reuse. A detailed case study on the reuse of fifteen bridge foundations primarily using Options 3 and 4 is presented later in this chapter.

INTRODUCTION

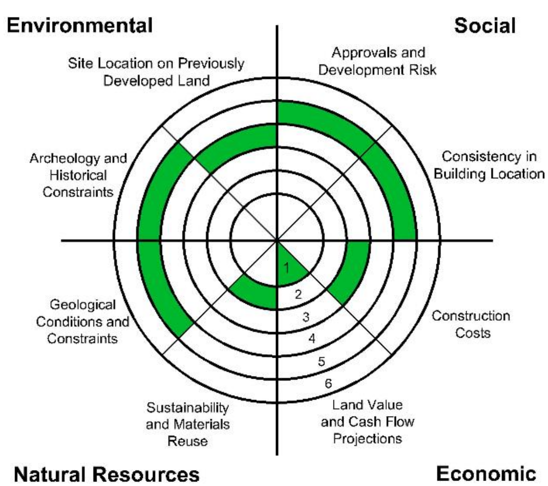
The 2017 National Academy of Engineering (NAE) report on "NAE Grand Challenges to Engineering in the 21st Century" has identified "Restore and Improve Urban Infrastructure" as one of the 14 grand challenges (NAE 2017). As per 2016 National Bridge Inventory data, 9 percent of bridges (56,007 out of 614,387) are in poor condition, 15 percent are older than the average design life of 70 years for bridges and almost 9 percent are in urgent need of rehabilitation or replacement (FHWA 2017). Replacement or significant rehabilitation of these bridges, particularly those in urban areas, is challenging because of mobility and traffic demands. Management of aging infrastructure is therefore one of the most important challenges for transportation agencies across the United States and other parts of the world. As this infrastructure ages, existing bridges will need to be rehabilitated or replaced, depending on the level of deficiency. In many cases, bridges will require superstructure replacement, while the foundation still has significant functional value.

Reuse of bridge foundations can result in significant time and cost savings. Foundation reuse is particularly attractive for bridges in densely populated urban areas or on important routes where a complete or partial closure of a bridge may lead to severe congestion and mobility problems. Reuse of a bridge foundation can also be an attractive option for historical preservation of a bridge or surrounding landscape. Foundation reuse offers numerous unique economic, social, and environmental incentives that can be leveraged to facilitate rapid superstructure replacement through accelerated bridge construction (ABC). Another strong justification for reuse may be made on the basis of proven performance of the foundation during its initial service life and its potential for additional life. Tremendous benefits of foundation reuse have already been demonstrated through numerous bridge superstructure reconstruction projects executed in different parts of United States during last 10 years.

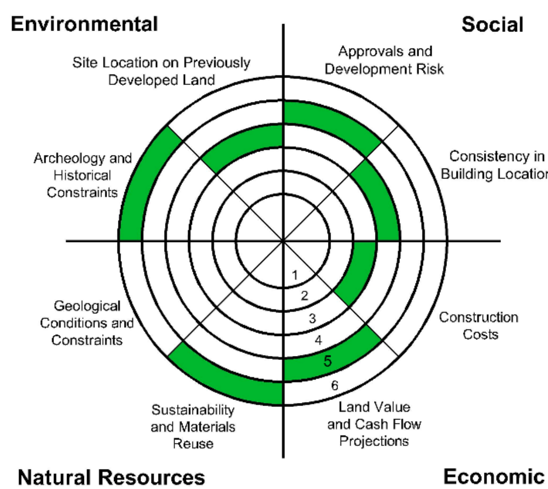
Despite the benefits, the reuse of bridge foundations presents many challenges and complex decision-making processes. The recent NCHRP Synthesis 505 "Current Practices and Guidelines for the Reuse of Bridge Foundations" (Boeckmann and Loehr 2017) has presented a comprehensive literature review as well as a survey of transportation agencies in all 50 States, Puerto Rico, and Canada. The synthesis found that major challenges to reusing bridge foundations include: understanding of the current condition of the foundation, determining the load-carrying capacity of the foundation, estimating the remaining service life of the foundation, and evaluating the reused foundation using the guidelines and codes developed for new foundations. Addressing these issues in any bridge project requires thorough investigation of the substructure and foundation. This report manual presents technical information on all aspects of the decision process leading to foundation reuse. This includes the drivers for foundation reuse, integrity assessment, durability assessment, service life estimation, foundation capacity assessment, and strengthening/enhancement alternatives. Discussions on these topics have been supplemented by case studies from foundation reuse projects carried out in United States and Canada.

In 2003, the European Union started a research project called RuFUS to study the feasibility of foundation reuse in buildings and published "A Best Practice Handbook of Reuse of Foundations for Urban Sites" (Butcher et al. 2006). The potential for cost and time savings through reuse were investigated along with technical and liability issues associated with reuse projects. Case examples and flowcharts were provided to show the application of the discussed procedures and methods. Furthermore, the Construction Industry Research and Information Association (CIRIA) in London, U.K., published a guide on reuse of buildings foundation (Chapman et al. 2007). This guide discusses practical and technical issues related to reuse of an existing building foundation for new construction and proposes approaches to reduce the associated risks.

Eight drivers for foundation reuse in buildings were investigated using the Sustainable Project Appraisal Routine (SPeAR®) diagram shown in figure 2 (Strauss et al. 2007). These eight drivers are: (i) site location on previously developed land, (ii) archeology and historical constraints, (iii) geological conditions and constraints, (iv) sustainability and materials reuse, (v) land value and cash flow projections, (vi) construction costs, (vii) consistency in building location, and (viii) approvals and development risks. Each of these drivers is represented by a segment in the SPeAR® diagram and assigned a rating from 1 to 7, with lower score indicating greater value for reuse over replacement. The appeal of reuse over replacement increases as the diagram for a site resembles a bull's eye more closely. Figure 2 shows modified SPeAR® diagrams for New York and Houston provided by Strauss et al. (2007) that suggest reuse of building foundations is more favorable for a specific New York site than a specific Houston site.



A. New York City site



B. Houston, TX site

Figure 2. Diagram. Modified SPeAR® diagram method applied to two building sites in United State cities

Although the reuse of both bridge and building foundations may be motivated by many similar drivers, the reuse of bridge foundations may require more extensive evaluation than reuse of building foundations. Bridge foundations typically have greater exposure to adverse environmental conditions and are subjected to a wide range of loads due to hydraulic scour, wind, earthquakes, traffic loads, impacts, and so forth.

In the United States, the earliest effort toward developing a knowledgebase on foundation reuse was initiated by the North Carolina Department of Transportation through a polling of 10 State transportation agencies regarding their experience with foundation reuse. The results were presented during an FHWA workshop in 2013 under the Foundation Characterization Program (Collin and Jalinoos 2014). As per this limited study, seven agencies had reused an existing foundation for bridge replacement, eight had retrofitted an existing foundation, and all 10 had experience with one or the other. Despite this, none of the agencies reported having policies or guidelines on foundation reuse in place. However, several State Departments of Transportation have at least some provision in their bridge design manuals referencing foundation reuse.

The Illinois Department of Transportation (IDOT) was an early adopter of a written foundation reuse policy, with reuse procedures in place since the late 1980s. IDOT's current policies related to foundation reuse are incorporated in the agency's Bridge Condition Report Procedures and Practices (IDOT 2011). The report outlines development of bridge condition reports that identify the current condition and functionality of a bridge to establish a scope of work for eventual bridge rehabilitation or replacement. The IDOT report also provides detailed guidelines for reuse of different bridge components, including superstructure (decks/slabs, girders/beams, bearings) and substructure (caps, columns, stems, footings, foundations) components. For substructure components, the guideline requires evaluation of substructure condition, load capacity, and scour assessment, if applicable. The main load-carrying elements of the substructure are evaluated to determine if they are structurally sound and in sufficient condition to remain in place, or if they require repair or replacement. Main load-carrying structural elements often consist of pier caps, columns, pier stems, abutments, footings, and piling. Some areas of importance considered during the evaluation are significant section loss, damage that affects capacity, and deterioration levels that indicate possible reduced capacity. IDOT (2011) also provides guidelines on evaluating the capacity of existing foundation elements (piles and spread footings), rather than using the original design capacity. An increase in geotechnical resistance of piles due to their prior service history is considered during the evaluation of the capacity of existing piles. Compliance of existing foundations being reused to the LRFD Bridge Design Specifications (AASHTO 2014) is also discussed, along with the potential for reusing deficient substructure by either reducing new dead loads or by strengthening substructure components.

The Maine Department of Transportation first included foundation reuse guidelines in the 2003 version of its *Bridge Design Guide* (MaineDOT 2014) in Chapter 10, entitled "Bridge Rehabilitation." This guideline recommended performing life-cycle cost analysis to decide if rehabilitation is preferred over replacement. Section 10.7, titled "Substructure Reuse," of this guideline provides the most pertinent information to foundation reuse. The section calls for the evaluation of existing foundations whenever the substructure is to be reused "with new loads applied."

The LRFD Bridge Manual of the Massachusetts Department of Transportation (MassDOT 2013) includes decision methodologies and evaluation criteria for designers to use when determining the scope of repairs and replacements required for each bridge project. The evaluation criteria are clear, but the manual affords agency engineers judgment to interpret the criteria.

Aktan and Attanayake (2015) investigated the evaluation and standardization of accelerated bridge construction techniques for the Michigan Department of Transportation (MDOT). This report discusses foundation design, construction, and upgrade methodologies for an in-service bridge, with a specific focus on reuse. Work presented in this report can be used for scoping bridge foundation reuse projects and determining available alternatives.

Boeckmann et al. (2018) evaluated the integrity and capacity of driven piles at two Missouri DOT bridges slated for demolition. The driven piles at one bridge consisted of closed-end steel pipe piles backfilled with concrete (CIP piles), and the other by octagonal precast concrete piles. Parallel seismic (PS) and sonic echo/impulse response (SE/IR) testing was performed at both sites prior to exhumation of the piles. Pile lengths estimated with PS ranged from ±17 percent of the length measured from the exhumed piles. SE/IR was not effective on piles prior to demolition of the bridge but provided a similar ranged estimate as PS testing after demolition of the bridge. Visual inspection was used to identify corrosion extent on the steel piles, with cross-sections taken to measure penetration. Only minor corrosion with minimal penetration was observed. Static load tests and restrikes with a pile driving hammer (for dynamic capacity estimation) were performed on select in-situ piles to compare the in-situ capacity with the design plan capacity and capacities estimated using empirical static capacity estimations. The capacities from static estimation, static testing, and CAPWAP testing during restrikes were in good agreement and exceed the design plan values by a factor of 5 to 10.

MOTIVATIONS

Reuse of bridge foundations potentially offers numerous economic, environmental, and social benefits that can drive the decision-making process. In many cases, competing replacement/reuse alternatives may offer various benefits that may be difficult to objectively compare and quantify. To have a better understanding of the advantages of foundation reuse, the underlying drivers can be lumped into three categories: economic, environmental, and social. These three categories form the "triple bottom line" of sustainability described by the INVEST program (FHWA 2017), as shown in figure 3. The individual benefits derived from foundation reuse can impact one or more of these categories.

Figure 3. Diagram. Three drivers for foundation reuse (Adapted from FHWA INVEST website)

NCHRP Synthesis 505 (Boeckmann and Loehr 2017) presented a survey of over 62 transportation agencies (50 U.S. DOTs, Puerto Rico, and the District of Columbia, as well as ten Canadian provinces and territories). Economic considerations, accelerated construction, constructability, project schedule, environmental/permitting considerations, bridge repurposing, right of way constraints, historic preservation, emergency repairs, and utility conflicts were identified as key motivations for foundation reuse, as shown in figure 4. These motivations can be grouped into six drivers for foundation reuse: costs (agency and user costs), sustainability (resources and asset management, social and environmental impacts), site constraints (right of way constraint, constructability, and utility conflicts), mobility, historic/aesthetic preservation, and environmental impacts and permitting. These drivers can be assigned importance weights based on the survey results in figure 4 and can be plotted as modified SPeAR® diagram (Laefer 2011) in figure 5. These six drivers are assigned weights in figure 5 from 1 to 6 (note that sustainability is shown twice, as an economic and environmental driver). The SPeAR® diagram in figure 5 may be plotted for various reuse and reconstructions options at a bridge site (possibly from multiple of the Options 1 to 4 in figure 1) to display the overall attractiveness of an alternative based on the six drivers. The SPeAR® diagram with highlighted areas closest to the bull's eye will represent the most attractive option considering these six drivers.

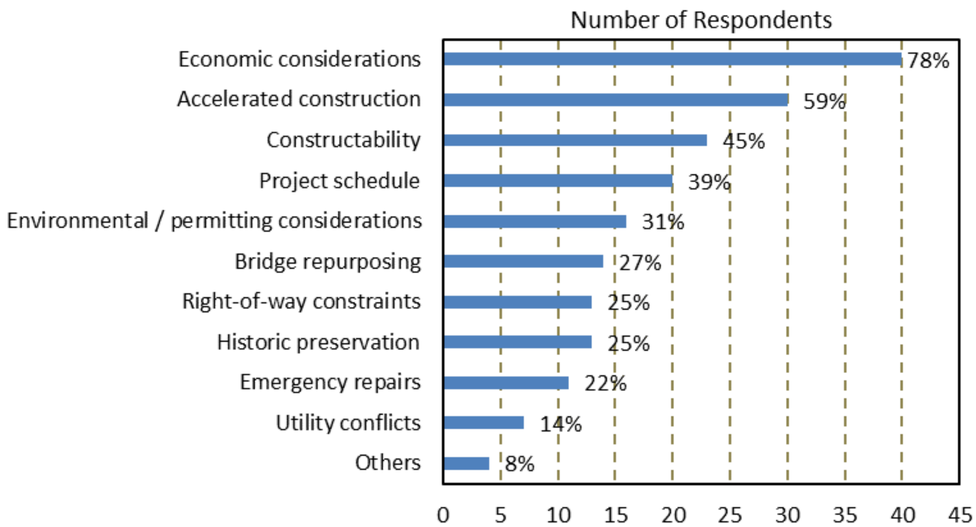


Figure 4. Graph. Survey on motivations for foundation reuse

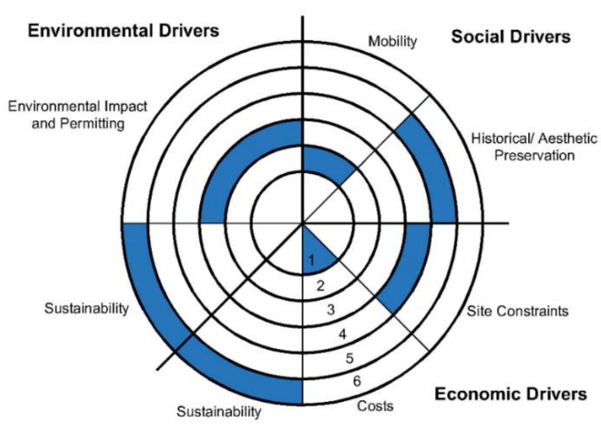


Figure 5. Diagram. Modified SPeAR diagram for bridges. (Adapted from Laefer 2011)

Economic Drivers

The results in figure 4, from NCHRP Synthesis 505 (Boeckmann and Loehr 2017), show that economic considerations were the most common motivation, with 40 agencies (78 percent of responses) reporting them as a factor. Direct cost savings due the reuse of an existing foundation can be achieved during all three phases of a project:

- Planning: Complete reconstruction projects (Options 1 or 2 in figure 1) can incur significant planning costs that may be avoided by selecting a reuse alternative. Option 1 alternatives often require planning for changes to the bridge alignment and acquisition of the right-of-way. Option 2 alternatives may require additional planning for construction of the new elements, demolition of old elements, and sequencing. Both Option 1 and 2 alternatives potentially require additional planning costs associated with the environmental impact of demolition and construction of new components. On the other hand, reuse alternatives (Options 3 and 4) use the existing alignment and right of way, while reducing the amount of construction/demolition and environmental impact. Reuse alternatives also may require significantly lesser planning for traffic management.
- Design: Reconstruction alternatives require complete design of a new foundation, which may include additional geotechnical investigation costs, type selection costs, environmental protection costs, and realignment costs. Reuse alternatives may incur costs associated with the assessment of integrity, durability, capacity and remaining life of existing foundation components. Hence, design costs for reuse alternatives may be higher than those for reconstruction alternatives. However, these additional costs may be insignificant compared to savings during the construction phase.
- Construction costs: Foundation reuse allows for potentially large construction cost savings by reducing the amount of required materials and work. Option 1 or 2 alternatives may require constructions adjacent to or near in-use structures that increases construction costs beyond typical levels.

Foundation reuse presents potential savings obtained in all three phases when compared to complete reconstruction. The reconstruction of the Milton-Madison Bridge between Kentucky and Indiana realized a total cost saving of $50 million from reuse of substructure and foundation components (Tiberio 2015). The reuse of the Lake Mary Bridge foundations and substructures will result in saving of $500,000 in construction cost and three months in construction time (CFLHD 2015). Detailed information on these projects is provided in the “Case Examples” section of this chapter.

Environmental Drivers

Environmental concerns were mentioned as a reuse motivation by 31 percent of transportation agencies during the NCHRP survey. Significant environmental benefits derived from foundation reuse include a reduction in the following:

- Construction materials usage (steel, concrete, wood).
- Energy consumption (demolition, materials production, transportation, and construction operations).
- CO_2 emissions (materials production, transportation, and construction operations).
- Impact on natural resources (land acquisition, water resources, habitat restoration, and site vegetation impacts).

- Waste generation (demolition of existing foundation components).
- Construction noise.
- Construction time (Accelerated Bridge Construction).

Social Drivers

For any project executed on in-service infrastructure, closure time is often the most important considerations. Solutions that enable agencies to reduce the amount or improve the timing of closures can decrease user costs of a project significantly. Accelerated bridge construction (ABC) is a newly implemented approach by transportation agencies to achieve this goal of reducing user costs during reconstruction or rehabilitation of a bridge. Option 1 alternatives often allow preservation of traffic flow during much of the construction and require closure during roadway alignment or bridge replacement. When replacement foundation elements are very close to the existing elements, there may be traffic impacts from the construction. Option 2 bridges may require closure of the bridge during significant portions of the construction period if the new foundation elements cannot be constructed while the bridge is in service. If new foundation elements can be constructed for Option 2 alternatives while avoiding the existing foundation (i.e., constructing a new foundation away from the existing foundation in the existing alignment), accelerated bridge construction (ABC) methods can be used to rapidly replace the superstructure. Bridge closure in this case may be minimized through proper planning. Reuse (Option 3 and 4) alternatives often allow preservation of the existing right of way, reducing delays associated with changing road alignment.

For bridges of historical significance, notable architectural styles, and/or aesthetic appeal, retention of the original appearance of the bridge can carry significant social value. However, it may be necessary to retrofit or rehabilitate reused components to address integrity, durability, and capacity concerns. Foundation reuse is often an attractive option in such cases, as demonstrated by the North Torrey Pine Bridge in California (Johnson and Creveling 2015) and the Henley Street Bridge in Tennessee (Das 2010).

APPLICATIONS OF FOUNDATION REUSE

In the survey presented in NCHRP Synthesis 505 (Boeckmann and Loehr 2017), State DOTs and other agencies were asked about applications for which foundations were reused. Table 1 shows the results of this survey. Bridge widening, and replacement were by far the most common applications for reuse, with 41 respondents (82 percent) and 40 respondents (80 percent), respectively. Seismic retrofit, increasing clearance, and bridge repurposing were less common applications, but each was selected by at least 24 percent of respondents.

Table 1. Applications for foundation reuse (Boeckmann and Loehr 2017)

Response	Number	Percent
Bridge or superstructure widening	41	82
Bridge or superstructure replacement	40	80
Seismic retrofit of bridge foundations	15	30
Increase clearance (e.g., over railway)	15	30
Bridge repurposing	12	24
Other:		
Scour retrofit	1	2
Cantilever retaining wall	1	2

CHALLENGES

In 2013, the FHWA's Foundation Characterization Program hosted a stakeholder workshop to discuss program goals. Participants included the FHWA, representatives from five state transportation agencies, academia, and industry. The participants identified four major challenges for reuse of existing bridge foundations:

1. **Condition assessment:** What is the structural integrity of the existing foundation?
2. **Load capacity:** How can capacity of the existing foundation be determined, and is it greater than the value from the original design?
3. **Remaining service life:** How much longer can the existing foundation be expected to maintain serviceability?
4. **Design codes:** How should existing foundations be considered within the context of established design codes and specifications, which were developed for new foundations?

These issues were also identified by NCHRP Synthesis 505 (Boeckmann and Loehr 2017). An additional challenge is an uncertain standard of care for consulting engineers involved in foundation reuse (Brown 2014).

SUBSTRUCTURE ISSUES IN ABC PROJECTS

Accelerated bridge construction (ABC) has been increasingly used in highway construction projects to decrease construction time and reduce mobility impacts. To facilitate rapid construction, ABC projects often reuse portions of the substructure to support a new superstructure that is prefabricated then lifted, transported, or slid into position. Much of this construction is performed while the existing bridge is in service, thereby requiring special loading considerations for superstructure as the replacement bridge is being constructed, as the original superstructure is removed, and as the new superstructure is being positioned. For slide-in bridge construction (SIBC), a temporary substructure is often built to support the superstructure prior to sliding it onto the existing substructure. Bridges can also be transported into place using a self-propelled modular transporter (SPMT) that carries the bridge into place. ABC projects are commonly completed using all options presented in figure 1.

In Option 1, bridges are constructed on an alignment adjacent to the existing roadway alignment. The new foundations and deck are typically constructed while the existing bridge is in service, thereby limiting impacts on user mobility. The process of switching from the original alignment to the new alignment can impact user mobility, though closures can be limited to a short time. Option 2 preserves the existing alignment by constructing new foundation elements within the existing alignment. Substructure construction is often completed while the existing bridge is in service, although sometimes requiring lane closure or nighttime construction for installation. After the new substructures are completed, the new bridge can be slid, lifted, or transported into place using as little time as a weekend. Option 2 designs that require demolition of existing elements prior to construction of the new substructure will cause significant mobility impacts unless traffic is diverted to a temporary bridge. Option 2 alternatives may include straddle bents (when a new drilled shaft is constructed outside the bridge footprint while a bridge is in service).

In Option 3, foundations and substructures are typically investigated and repaired (if necessary) prior to the removal of the superstructure and traffic closure. The new superstructure is then placed on the existing foundations shortly after removal of the old superstructure. In Option 4, foundations and substructures are typically repaired/strengthened prior to the removal of the superstructure. When this is not possible, the use of prefabricated bridge elements and systems (PBES) allows for large reductions in the amount of closure time. Table 2 presents a selection of ABC case studies found in the FIU UTC ABC project database (FIU 2017) that involved substantial substructure work or planning. The cases that involved reuse of the existing substructure are italicized.

I-95 Corridor Replacement Project, VA

The I-95 corridor replacement project involved replacement of 11 aging and deteriorated bridges located near the junction of I-95 and I-64 (LeGrand 2015; Jalinoos et al. 2016; Jalinoos 2015). These bridges were subjected to very high traffic volumes of 150,000 vehicles per day (LeGrand 2015) that could not be accommodated through diversion to local roads or other highways. Bridge closure for any significant period was highly undesirable due to high traffic volumes. ABC using preconstructed bridge units was chosen as it could deliver the project while only requiring bridge/lane closures on weekday nights between 8 p.m. and 6 a.m. The bridge units were constructed off-site and transported to each of the 11 bridge sites. Upon arrival, the original bridge superstructure was demolished, and the new deck was placed on the existing substructure elements, which were reused without requiring any strengthening. The new decks were constructed from lightweight concrete that lowered the loading on the substructure by approximately 7 percent (Jalinoos 2015). The substructures were analyzed to find their LRFD capacity, which was sufficient for the new loading. Three of the bridges in the corridor were widened during this process, requiring the installation of 50 additional drilled shafts (LeGrand 2015). The new drilled shafts were installed outside of the original pier footprints prior to placement of the new bridge decks. Many of the substructures had undergone varying amounts of damage during their original lifespan and were repaired appropriately prior to the deck replacement.

Table 2. Notable ABC projects, with the ABC method and substructure issues

Bridge Name	ST	Placement Methods	Opt.	Substructure Work/issue
I-5/U.S. 12 Bridge at Grand Mound	WA	CHLE[1]	1	Complete replacement, precast pier columns connected to cast-in-place footings.
Sprain Brook Parkway Bridge over NY Route 119	NY	CHLE	2	Single span bridge with MSE concrete abutments and wingwalls. New abutments built outside of existing abutments while bridge was in service
Craig Creek Bridge	CA	CHLE	2	Steel piles filled with concrete (CISS piles) driven in roadway, then abandoned and paved over. During next construction season, temporary bridge placed parallel to roadway, piles excavated, and a precast abutment and deck placed on piles.
Well Road Bridge	*LA*	*SPMT*[2]	*4*	*Existing substructure strengthened by adding concrete footing between existing concrete drilled shafts. Abutments were widened using additional drilled shafts*
TH 61 Bridge over Gilbert Creek	MN	CHLE	1	Precast pier and abutments on driven pipe piles
I-44 Bridge over Gasconade River	*MO*	*SIBC*[3]	*3*	*Existing substructure repaired and reused. Temporary substructure built to hold bridge before slide*
640th Street Bridge over Raccoon River	IA	CHLE	2	10 piles driven in 1 day after removal of bridge deck. Piles driven with tight tolerance to fit into pockets in precast abutments
Riverdale Road Bridge over I-84	UT	CHLE	1/2	Two new bridges built along either side of existing bridge. Traffic was diverted to new bridges, Precast abutments and piers post tensioned to existing abutments during reuse
U.S. 131/Parkview Avenue Bridge	MI	CHLE	2	Abutments precast in 2 pieces connected with closure pours, supported on cast in place footings.
I-215/4500 South Bridge	UT	SPMT	2	Abutments replaced in front of existing abutments, new deck placed on abutments during a single weekend closure. Lightweight concrete was used in the deck to reduce loading and eliminate 3 center piers (4-span bridge replaced with single span).
I-80 Bridge over Dingle Ridge Road	NY	SIBC	1	New abutments on drilled shafts added in front of existing abutments on shallow footing, existing piers reused
OR Route 38 over Elk Creek	OR	SIBC	2	Temporary substructure built to hold new superstructure during construction of new substructure. Single weekend closure while new deck was slid onto existing alignment.
U.S. Route 4 over Ottauquechee River	*VT*	*CHLE*	*4*	*Existing substructure encased while in service. Old bridge removed and replaced with new prefabricated elements.*
Cedar Street Bridge	*MA*	*SPMT*	*3*	*Full substructure reuse following capacity analysis from driving logs without test data*
I-95 corridor bridge replacement	*VA*	*CHLE*	*3/4*	*11 bridges replaced with full substructure reuse. 9 bridges installed with cathodic protection, 1 with ECE, 1 with ECE and cathodic protection*
Fast-14 replacement project	*MA*	*CHLE*	*3*	*14 bridges replaced on original substructures with minimal repairs*

[1] Conventional heavy lifting equipment (CHLE)
[2] Self-propelled modular transporter (SPMT)
[3] Slide-in bridge construction (SIBC)

In general, the substructures were found to be in relatively good condition, although almost all of them had undergone at least some chloride-related corrosion. The main source of damage was chloride intrusion, which had led to reinforcement corrosion, cracking, and spalling. VDOT established a criterion to replace an element when the cost of all required repair, electrochemical extraction (ECE), sealer, and cathodic protection exceeded 40 percent of the replacement cost of the element. A threshold of 0.08 percent chloride content by weight at the reinforcement level was established. Elements with a higher chloride content than 0.08 percent were deemed to require ECE in addition to cathodic protection and sealer.

In all, 2 of the 11 bridges exceeded this threshold and required ECE. Patching of spalls and cathodic protection with an aluminum-zinc-indium galvanic anode was applied to all nine bridges that did not require ECE. To help evaluate the effectiveness of the combination of ECE and cathodic protection over time, cathodic protection was not applied to one of the two bridges where ECE was performed. Overall, it was expected that ECE alone would extend the usable life of the substructures by 15 to 20 years. ECE, sealers, expansion joint changes, and cathodic protection could extend the usable life of the piers significantly longer (Sharp 2016). A photo of the repaired foundation with a replacement superstructure is provided in figure 6.

Source: Virginia Transportation Research Council

Figure 6. Photo. I-95 Bridge over Overbrook Road Showing Underside after renovation

Fast-14 Replacement Project, MA

The I-93 corridor just north of Boston through Medford, MA carries over 200,000 vehicles per day over four traffic lanes in each direction (Moran 2012; Jalinoos et al. 2016). This corridor contained seven bridges in each direction that had undergone extensive deterioration since their construction over 50 years ago. Patching and repair work had become increasingly frequent and MassDOT decided that the superstructures to these bridges had to be entirely replaced. Since this highway is the only major interstate in and out of Boston from the north, redirection of traffic during construction was considered infeasible. Hence, ABC was pursued to allow the bridges to be replaced only during weekends, where traffic would be reduced to two lanes in each direction and diverted over to one side of the highway while the bridges on the opposing side were replaced. All 14 bridges were eventually replaced in a total of 10 weekend diversions, without any complete closure of the highway traffic. The replacement bridge decks consisted of steel girders with composite concrete decks that were prefabricated off-site and moved into place. Aside from normal deterioration, the substructures were considered to be in "salvageable" condition (Moran 2012). MassDOT opted to repair the observed deterioration and reuse the substructures as is. Changes were made to the expansion joints of the bridge to prevent water seepage that had caused damage to the original decks and substructures. A photo of demolition of one of the original bridge decks is provided in figure 7.

Source: FHWA

Figure 7. Photo. Demolition stage for removing the existing girders

CASE HISTORIES OF FOUNDATION REUSE

Several case examples involving foundation reuse are discussed in this section to highlight different aspects of foundation reuse. More detailed information related to the integrity, durability, and capacity assessment, as well as strengthening and repair measures undertaken is provided in the case study sections in subsequent chapters.

Table 3 below presents different foundation construction options used in these bridges. For 14 of the 15 bridges, either Options 3 and/or 4 were used. For the Hurricane Deck Bridge in MO, Option 3 was found to be viable, but a total replacement (Option 1) with significantly lower superstructure costs (leading to a slightly lower overall cost) was adopted. In addition to the case studies discussed herein, NCHRP Synthesis 505 (Boeckmann and Loehr 2017) describe additional case histories, including the Bowker Overpass in Boston, MA and Hunt Road over I-495 in Lowell, MA. The Foundation Characterization Program (FCP) TechBrief No. 1 (Collin and Jalinoos 2014) presents seven case histories, including three discussed in this report: The Hurricane Deck Bridge (MO), the I95 corridor bridges (VA), and the Henley Street Bridge (TN). Four case histories discussed in Collin and Jalinoos 2014 that are not included in this report are Bridge B-23-005-M-18-002 (MA), the Arthur Mills Bridge (UT), the Clipper City Rail Trail (MA), and the Yadkin River Bridge 91 (NC).

Table 3. Foundation reuse case histories

Case History	Owner	Option No.	Purpose	Primary Drivers for reuse	ADT[1] (VPD)
Lake Mary Bridge	Arizona DOT	3	Widening	Economic	750
Milton Madison Bridge	Kentucky/ Indiana	4	Superstructure Replacement	Economic/ABC	10,000
North Torrey Pines Bridge	Del Mar City	4	Rehabilitation	Historical	19,173
Georgia Street Bridge	City of San Diego	4	Seismic Retrofit	Historical	1,100
Huey P. Long Bridge	Louisiana DOT	4	Widening	Economic	45,000
Manahawkin Bay Bridges	New Jersey DOT	3	Widening	Economic	25,500
U.S. Route 2A Bridge	Maine DOT	3	Superstructure Replacement	Economic	442
U.S. Route 1 Viaduct	Maine DOT	3	Superstructure Replacement	Economic/ABC	17,974
Mississagi River Bridge	The Ontario Ministry of Transportation	4	Superstructure Replacement	Economic	3,920
Henley Street Bridge	Tennessee DOT	4	Widening/ Rehabilitation	Historical	43,230
Jackson Road over MA Route 2	Massachusetts DOT	4	Superstructure Replacement	Economic/ABC	3,900
Cedar Street over MA Route 9	Massachusetts DOT	3	Superstructure Replacement	Economic/ABC	15,000
Bay of Quinte Skyway	The Ontario Ministry of Transportation	3	Superstructure Replacement	Economic	5,290
Crowchild Trail Bridge	The Alberta Ministry of Transportation	3/4	Widening	Economic/ Environmental	107,000
Hurricane Deck Bridge	Missouri DOT	1	Superstructure Replacement	Economic	8,166

[1] Average daily traffic (Vehicle per day)-the most recent data (source: NBI database, MTO)

Table 4 summarizes technical information about the case history bridges (year built, number of spans, superstructure and foundation type), and lists the chapters in this report in which they are addressed.

Table 4. Summary of case history bridges

Bridge Name	Year Built	Number of spans	Superstructure Type	Foundation Type	Chapter Addressed
Lake Mary Bridge	1935	3	Steel girders-Continuous	Masonry Footings	4,6
Milton Madison Bridge	1928	4	Truss	Caissons	4,5,6
North Torrey Pines Bridge	1933	13	Concrete girders	Footing and Piles	4,5,6
Georgia Street Bridge	1914	1	Concrete arch	Spread Footing	4,5,6
Huey P. Long Bridge	1935	4	Truss	Caissons	4,6
Manahawkin Bay Bridges	1962	3	Concrete girders	Timber Pile	4,5,6
U.S. Route 2A Bridge	1953	3	Steel girders	Timber Pile	5,6
U.S. Route 1 Viaduct	1957	20	Steel girders	Footings and Steel Piles	4,5,6
Mississagi River Bridge	1943	5	Steel girders	Spread Footing	4
Henley Street Bridge	1930	6	Concrete arch	Spread Footing	4
Jackson Rd over MA Rt 2	1951	2	Steel girders	Pile	4,6
Cedar Street over MA Rt 9	1932	2	Steel girders	Footings	6
Bay of Quinte Skyway	1967	17	Steel girders	Caissons	-
Crowchild Trail Bridge	1966	10	Steel Girders	Footing/Pile	4,6
Hurricane Deck Bridge	1930	5	Truss	Caissons	-

Lake Mary Bridge Investigation, near Flagstaff, AZ

The Lake Mary Bridge (figure 8) is a three-span, 104-ft (32-m) long, and 34-ft (10-m) wide bridge, with a cast-in-place deck supported by steel rolled girders (Jalinoos 2015). The bridge was originally constructed in 1934 and widened in 1968. A 2014 Bridge Inspection report evaluated the deck as being in fair condition. Concrete spalling and exposed rebar at the suspended slab joint locations were mentioned in the inspection report. The steel girders were rated as poor and isolated corrosion was observed. The 2014 load rating reported inventory and operating ratings of 23.4 tons and 39.6 tons, respectively.

The Lake Mary Bridge was considered by the FHWA Central Federal Lands Highway Division (CFLHD) for widening the deck and replacing the existing superstructure with a new one atop the existing mass-gravity masonry substructure units. A detailed evaluation of the bridge substructure was carried out to determine the condition of the masonry substructure system (CFLHD 2015; Jalinoos 2015). This investigation included detailed nondestructive and geophysical investigation of bridge components. Eight coreholes were drilled through the substructures from the deck (two in each of the abutments and piers) into the underlying rock. The following wireline logging runs were performed from each corehole: Acoustic Televiewer, Optical Televiewer, Full Waveform Sonic, Compensated Density, Electric Log with spontaneous potential (SP)/ single point resistance (SPR), and Caliper log.

Based on the Lake Mary Road Widening and Bridge scoring report (Jacobs Engineering 2013), the replacement of superstructure by considering the reuse of the exiting foundation was considered. Important issues during alternative selection included construction cost, construction duration, minimizing environmental impacts, increasing structural capacity and improving safety. The proposed replacement bridge superstructure is a three-span structure, approximately 107 feet long. Since the results of preliminary investigation showed that the existing masonry piers and abutments and foundations were in good condition, minimum modifications were proposed for them.

The Structure Selection Report, Lake Mary Road (CFLHD 2015) estimated that at least $500,000 in cost savings and three months in construction time could be realized through the foundation reuse.

Source: FHWA

Figure 8. Photo. Lake Mary Bridge in Arizona

Reuse of Milton Madison Bridge Piers, between Madison, IN and Milton, KY

The original Milton Madison Bridge was a historic 5-span through truss bridge that crossed the Ohio River between Indiana and Kentucky (Jalinoos 2015; Jalinoos et al. 2016; Tiberio 2015; Ligozio 2009). The bridge had a 20-ft-wide (6-m) deck comprising of two 10-ft-wide (3-m) lanes with no shoulder. This bridge did not meet current roadway width standards and was in poor condition. The bridge is a major crossing of the Ohio River that could create significant user costs when closed, and the bridge was adjacent to a historic district that could have been adversely affected by changes to the bridge alignment or style. The bridge was replaced in 2014 using slide-in-place construction on many of the original piers that required minimal bridge closures. Prior to reuse, the piers were evaluated to determine their material properties, current condition, remaining life, and strengthening measures required to achieve the necessary capacity and lifespan. The replacement bridge was widened to have a pair of 12-ft (3.7-m) wide traffic lanes, two 8-ft (2.4 m) wide shoulders, and a 5-ft (1.5 m) wide sidewalk on the downstream side of the bridge. A view of the original bridge is given in figure 9, while a rendering of the reconstructed bridge is given in figure 10.

Figure 9. Photo. Milton Madison Bridge prior to renovation

Figure 10. Photo. Milton Madison Bridge after renovation

North Torrey Pines Bridge Seismic Retrofit, Del Mar, CA

The North Torrey Pines Road Bridge in Del Mar, California was a 15-span precast concrete bridge originally constructed in 1933 (Jalinoos et al. 2016; Johnson et al. 2015a, 2015b). The bridge was considered a landmark bridge and was eligible for listing on the National Register of Historic Places (NRHP). The substructure contained a unique geometry consisting of 11 interior non-skewed bents, with another three highly skewed bents around a rail track underneath the middle of the bridge. There are two seat-type abutments at either end of the bridge; one with pinned connections to the superstructure and the other without lateral restraint. The interior bents range in height from 30 to 70 feet (9 to 21 m). The bridge had suffered extensive deterioration and was in poor condition. Concrete damage was visible throughout the substructure elements, although only a small percentage of the structure had spalled or delaminated. Due to high seismic demands and the old design, the bridge was considered susceptible to possible earthquake and liquefaction induced damage.

The results of a 2007 study showed the design service life of a retrofit would be a minimum of 50 years and cost approximately $30M, while replacement would have a minimum design life of 75 years and cost approximately $27M (However, final cost for the retrofit option was $21M). Despite being more expensive and having a shorter design life, the retrofit option was chosen because it preserved the historic appearance and the bridge's eligibility for the NRHP. A comprehensive seismic analysis was performed using both a nonlinear pushover analysis and a time history analysis. Seismic analysis of the as-built bridge revealed several issues that would present safety risks during the design seismic event including liquefaction, embankment lateral spreading, excessive shear and moment in pier columns, and excessive displacement demand in some of the columns. A significant retrofit program was undertaken to correct these issues, as shown in figure 11.

Figure 11. Photo. Installed column ties to increase the confinement of piers' columns

The existing foundations were evaluated for seismic performance using the loading calculated during a pushover analysis. Pile foundations were analyzed using a non-linear, pseudo-static pushover analyses and ultimate vertical bearing pressures were calculated using the Vesic (1973) bearing capacity formula for shallow foundations. The results of these analyses indicated the existing shallow and pile foundation capacities were adequate, except for the abutment foundations, which were found to provide inadequate restraint to the deck. The existing abutments were replaced with new integral abutments but left in place to maintain the historic appearance. Compaction grouting was performed to mitigate the liquefaction potential and seismic slope displacements.

Georgia Street Bridge Seismic Retrofit, San Diego, CA

The Georgia Street Bridge (figure 12) is a three-rib arch bridge, completed in 1914. In 1999, the bridge was listed on the National Register of Historic Places due to its historical significance in connecting the neighborhoods in eastern San Diego to downtown. The aesthetics and the functionality of the bridge were both impacted by deterioration of the concrete arch ribs, spandrels, and superstructure. The abutment and wingwalls had been repeatedly repaired over the years and were documented to be in poor shape. The abutments provided insufficient lateral restraint to the superstructure for the design earthquake loading. The focus of the rehabilitation program was to provide adequate lateral restraint to the abutments and wingwalls, to repair the observed concrete deterioration, and to meet modern seismic codes.

Figure 12. Photo. The Georgia Street Bridge and walls

Two replacement alternatives and two retrofit alternatives were considered. Although the cost of retrofitting the existing bridge exceeded the cost of both replacement alternatives, the historical significance of the existing bridge and site was an important consideration in this project. The Historic Property Survey Report (HPSR) showed that both bridge replacement alternatives, as well as retrofit alternative 2, would have had an adverse effect on the historic resource. Retrofit alternative 1 was evaluated as the most feasible design approach to have no adverse effect on the historic nature of the bridge.

Widening of the Huey P. Long Bridge, Jefferson Parish, LA

The Huey P. Long Bridge, in Jefferson Parish, Louisiana, is a combined railroad-highway bridge originally built in 1935. The bridge is listed as a National Historic Civil Engineering Landmark due in part to it being one of the longest high-level truss bridges in the United States (Modjeski and Masters 2013). The superstructure is supported by tall twin reinforced concrete columns that form gothic arches at the top. The original superstructure consisted of single 9-ft wide lines on either side of the rail right of way (ROW) that needed to be widened to support increased traffic demands. The Louisiana Department of Transportation and Development (LADOTD) considered reuse of the foundation to reduce costs related to environmental impacts, property acquisition, and construction. Significant savings were realized since the widening option remained within the existing foundation footprint, bridge right-of-way, and traffic corridors.

During the planning of superstructure widening, three superstructure options that utilized the existing foundations were investigated (cantilever widening, cable-stayed widening, and parallel truss widening). The selected option was parallel truss widening, where new trusses were installed outside of the original trusses and connected to widened portions of the piers using a steel transfer frame. The new trusses were connected to the existing trusses using new transverse elements, and the space in between the new and old trusses was used for the widened roadway. Figure 13 shows a view of the pier during widening, while figure 14 shows a view of the bridge after widening.

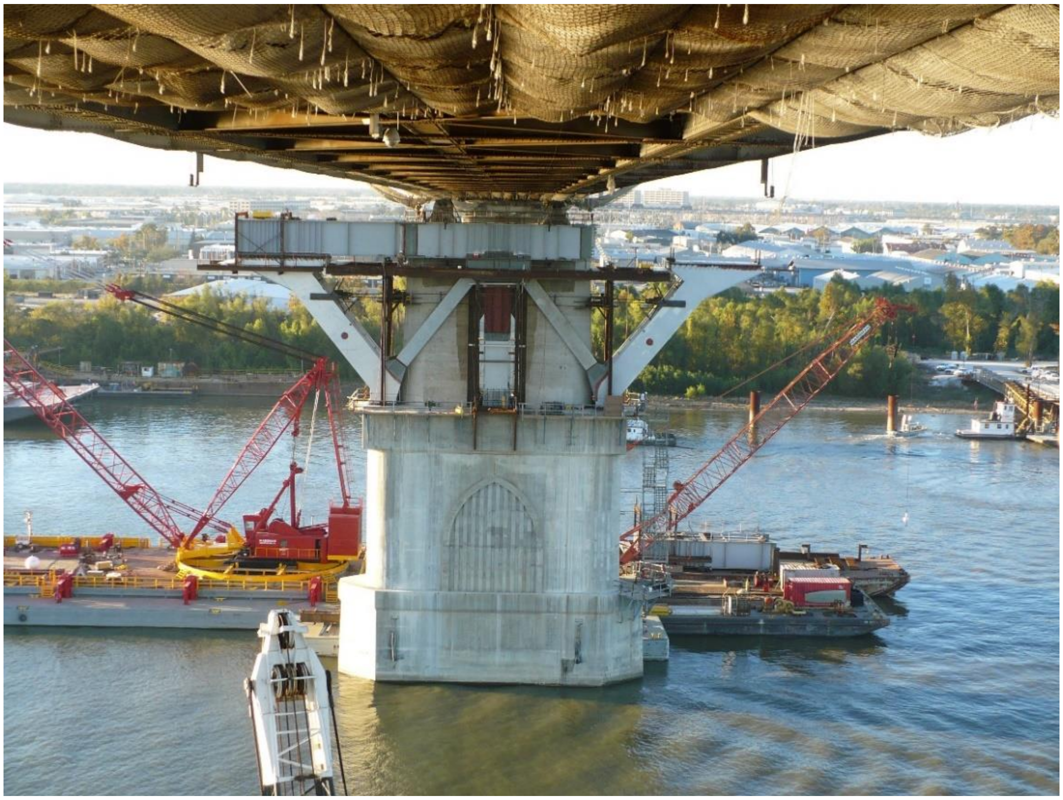


Figure 13. Photo. W-Frame under construction above encased pier

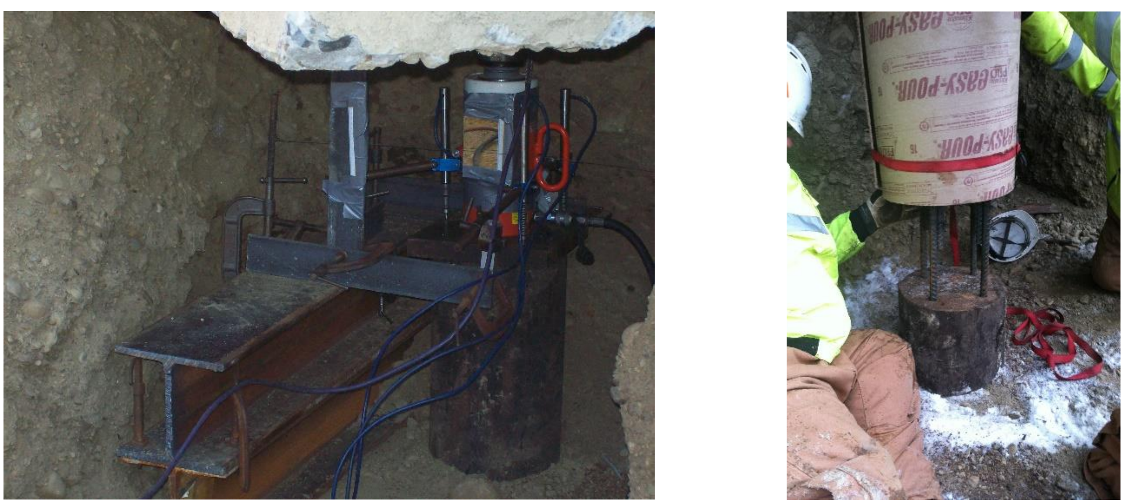
Figure 14. Photo. Huey P. Long Bridge with widened superstructure on reused foundation

NJ Route 72 Manahawkin Bay Bridges Project, Ocean County, NJ

The Manahawkin Bay Bridges carry NJ Route 72 over the Manahawkin Bay and connect Long Beach Island with mainland New Jersey. The bridges consist of three small trestle bridges and a longer span main bridge. The three trestle bridges have timber pile bents with a concrete pier cap, while the main span consists of concrete piers founded on timber piles. The bridges have suffered from increasing amounts of deterioration and traffic loads/volumes have increased, requiring rehabilitation of the bridges. Wider shoulders were desired to accommodate cyclists and emergency vehicles, and increased sidewalk width. The final construction plan involved the construction of a new bridge next to the existing main bridge to allow diversion of the eastbound traffic onto the new structure. The original main span, previously carrying traffic in both directions, will be repurposed to carry only westbound traffic. The new bridges will have the same number of lanes in each direction as the original bridge, but with increased lane widths, wider shoulders, space for cyclists, and a six-foot wide sidewalk on the north side of the bridge. The existing trestle bridge will be slightly widened to handle additional bike lanes and a sidewalk. The main span bridge was founded on timber pile, though these timber pile were permanently submerged and generally below the top of the existing ground surface.

U.S. Route 2A Bridge Foundation Reuse, Haynesville, ME

The Haynesville Bridge carries U.S. Route 2A over the Mattawamkeag River in Haynesville, Maine. The bridge is a 3-span steel girder bridge with two mass concrete stub abutments on treated timber piles and two solid wall piers on untreated timber piles. MaineDOT decided to replace the superstructure while reusing the substructures and pile foundations without modification. The piles were reused after a static load test was performed under each abutment. A resistance factor of 0.7 was applied to the nominal capacity of piles under the tested abutments, and 0.65 to all piles beneath the untested pier abutments, which is consistent with LRFD criteria. Figure 15 shows static load testing with a hydraulic jack after excavation of a pile cap and cutting of one of the timber piles.



A. Load test setup for north abutment B. Repairing of a test pile with concrete

Figure 15. Photo. Static load testing of existing timber pile

Figure 16 shows the results of the static testing on one of the test piles, along with its failure criterion. Both the loading and unloading of the test piles were recorded, and the permanent deflection was calculated. The original design capacity of 32 kips is shown, although this capacity was for allowable strength design and would be overly conservative for LRFD codes

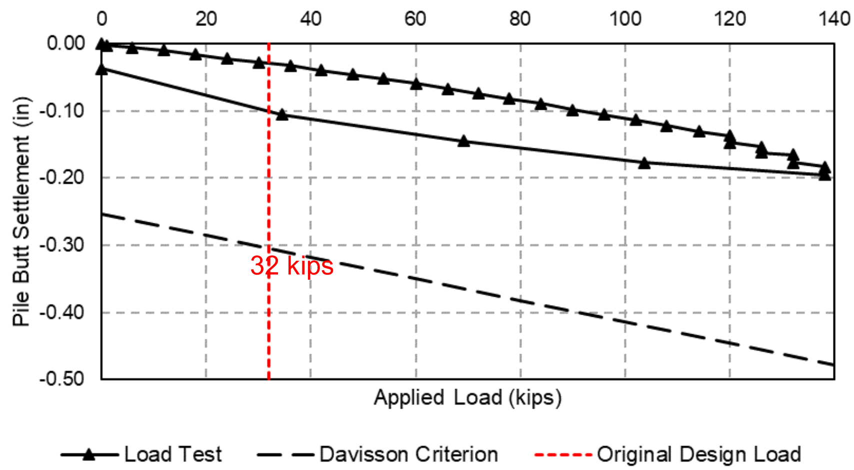


Figure 16. Graph. Load-deflection curve from static load tests of an existing timber pile in Haynesville

U.S. Route 1 Viaduct Reconstruction, Bath, ME

The 20-span viaduct that carries U.S. Route 1 over local streets in Bath, ME had undergone several rehabilitation efforts and the superstructure had experienced significant deterioration and needed to be replaced. The substructure elements including pier caps, abutments and piers showed substantial spalling and deterioration. An investigation was initially performed by MaineDOT to determine if the foundations could be reused during superstructure replacement, and whether or not 15 of the spans could be eliminated. Ultimately, the reconstructed bridge maintained the original geometry (no spans removed) while all above ground portions (piers, abutments, superstructure, etc.) were replaced. The bridge used prefabricated elements for the girders, pier caps, and pier columns that allowed the bridge to be demolished and constructed in only 195 days, even though the contract allowed for 220 days. The steel driven piles and/or footings to all nineteen piers and two abutments were reused.

In the bridge, 11 out of 19 piers and 1 of the abutments were supported by steel H-piles driven to bedrock. The other eight piers and remaining abutment were supported by spread footings on bedrock. The preliminarily estimated pile loads exceeded the factored axial resistance for most pier locations. Hence, the addition of micropiles was recommended. However, the final design of the bridge rehabilitation resulted in reduced loading that did not exceed the capacity of the H-piles. The bearing capacity of the piles was determined from subsurface investigation, review of as-built plans, and original construction logs. The consulting engineer recommended the structural resistance of the H-piles to be calculated with a reduced cross-section to account for potential corrosion during the intended 75-year service life extension. Parallel seismic nondestructive testing from a nearby borehole confirmed that the piles were approximately as deep as the bedrock, thereby providing confidence the piles had been driven to practical refusal on bedrock. Figure 17 illustrates the parallel seismic testing set up at the bridge site.

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Figure 17. Photo. Parallel Seismic (PS) setup for abutment of U.S. Route 1 Bridge in Maine

Mississagi River Bridge Rehabilitation and Deck Replacement, Ontario, Canada

The Mississagi River Bridge is a 5-span continuous girder bridge with a composite concrete deck supported by concrete abutments and piers with spread footings. (Li et al. 2014). The original bridge was constructed in 1943 and carried Provincial Highway 17 over the Mississagi River in Ontario, Canada. The bridge is the only major crossing of the Mississagi River for many miles in both directions and carried 3,920 vehicles per day as of 2011 (MTO 2011). Deck deterioration, steel element corrosion, and generally poor superstructure conditions were noted during previous inspections. The continuous girders were over loaded in the negative moment regions and fatigue prone details required frequent maintenance. Due to ROW acquisition constraints, the stakeholders preferred replacement of the superstructure with reuse of the existing piers.

The four concrete piers investigated had signs of reinforcement corrosion and spalling due in part to runoff through deck expansion joints. All four piers were believed to be founded directly on bedrock, although their depths and other installation details were not readily available. The pier footings had been originally installed using steel sheet piling used as cofferdams, with the concrete footings poured inside of the cofferdam after final excavation. An investigation consisting of coreholes through the piers, visual inspection, and ultrasonic inspection was performed. RQD measurements on extracted concrete cores showed recovery of less than 15 percent for three of the riverine piers, and 100 percent for the remaining pier. The coreholes were continued into the underlying soil, where three deteriorated piers were found bearing on a loose sand and gravel layer up to 10m above the bedrock, while the fourth pier was bearing directly on bedrock. Deterioration was observed on the steel sheet piling, and there was concern that the cofferdam was confining the soil underlying the non-bedrock footings and this corrosion could lead to loss of confinement and potential failure of the foundation. However, the substructures could be reused after strengthening with micropiles.

Henley Street Bridge Widening and Arch Repairs, Knoxville, TN

The Henley Street Bridge was a 6-span reinforced concrete arch bridge with a total length of 1793 ft (547 m), originally constructed in 1931. The bridge carried five lanes over the Tennessee River on a 54-ft (16.5-m) wide deck with two 6-ft (1.8-m) wide sidewalks, one on either side (Das 2010; Jalinoos et al. 2016). The bridge had significantly deteriorated and required strengthening and various repairs. There was also a desire to widen the bridge to allow more and wider traffic lanes. The bridge was historically significant due to its age and style, and was a signature crossing of the Tennessee River. To preserve the historic appearance of the bridge, reuse of the foundation, arches and spandrels was considered. The concrete overall was found to be in relatively good shape, although there were areas with very little concrete cover over the reinforcement where corrosion had already begun. Spalling had occurred around various construction joints where water ingress had impacted the reinforcement (Das 2010). The replacement bridge utilized the existing piers and arch ribs but replaced the deck and spandrel columns. The riverine piers were strengthened with additional drilled shafts that were assumed to take 80 percent of the load from the reused bridge.

Jackson Road Bridge Abutment Reuse, Lancaster, MA

The Jackson Road Bridge over MA Route 2 was constructed in 1951 and consisted of two 67-ft (20.5-m) steel girder simply-supported spans (GTR 2014). The substructure consists of two gravity concrete abutments and one concrete pier founded on timber piles. According to a 2012 bridge inspection report, this bridge was not in poor condition (did not require significant maintenance or rehabilitation to remain in service, not load posted), but it was in poor condition (lanes and sidewalks were narrower than current standards). To limit impacts on traffic management, ABC using precast elements was utilized for this project. To facilitate construction, the existing substructure components, including abutments, piers, and timber piles were reused.

Cedar Street Bridge, Wellesley, MA

The Cedar St Bridge, as shown in figure 18, was a 2-span continuous steel girder bridge that carried three lanes of traffic over MA Route 9 in Wellesley, MA. (Lamson Engineering 2011). The superstructure was in poor condition and in need of replacement, but the substructure had been rated in good condition in previous inspection reports. Full design drawing and as-built plans of the existing foundations showed that the foundation was supported by shallow footings on a sand and gravel layer. Since the substructure was in good condition, reuse was investigated to determine if ABC could be employed to replace the superstructure. A new superstructure constructed just off-site on temporary foundations and moved into place after the original structure had been demolished. This construction sequence allowed the contractor to limit road closures to only 72 hours. While the loads on the replacement substructure were not substantially changed from the original superstructure, the foundation had to be reanalyzed to ensure adequate capacity for construction and operational loading.

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Figure 18. Photo. Cedar Street over MA Route 9 Bridge

Bay of Quinte Skyway Superstructure Rehabilitation, Ontario, Canada

Quinte Skyway is a high-level 17-span bridge crosses the Tyendinaga Mohawk Territory in Ontario, Canada (Dow and Sokoloff 2013). As of 2011, the bridge carried an average of 5,290 vehicles per day (MTO 2011). The bridge was noted to be in deteriorated condition, and it was desired to provide an additional 40 years of service life. The bridge was considered to be a significant landmark and closure for construction would have required detours of up to 47km. Due to the cultural importance of the bay, it was required that no work be performed in the water, meaning that no foundation work was allowed. The existing piers are founded on caissons socketed into bedrock. The preliminary evaluation showed that reusing the caissons as economically, environmentally and socially beneficial. Figure 19 shows the landscape view of this bridge.

Figure 19. Photo. Bay of Quinte Skyway

Crowchild Trail Bridge, Alberta, Canada

The Crowchild Trail Bridge, in Calgary, AB, Canada crosses the Bow River (figure 20) with a 10-span bridge that also crosses rail lines, roads, and bike/shared use paths. The bridge carries a total of 107,000 vehicles per day (City of Calgary 2015). The bridge contains nine wall-type bridge piers with varying amounts of skew. Four of these piers are riverine although only three are submerged during low-flow conditions. The four riverine piers (Piers 1, 2, 3, and 4) are founded on concrete footings that are cast into the underlying bedrock (with the top of footing at the top of bedrock). Piers 5 and 6 are on land and are founded on pipe piles driven to bedrock, and the remaining three piers are on spread footings bearing on a gravel layer above the bedrock. The bridge is to be widened to provide one additional travel lane in each direction and to realign onramps at the ends of the bridge, where the land-based piers are located. The riverine piers were investigated for reuse due to the potential for more rapid construction, environmental concerns associated with drilling and construction in the river. These piers will be widened to support the additional travel lane width. The land-based piers will be reused and will be widened as needed to carry the additional loading from the wider superstructure. The riverine piers will not be widened and were instead analyzed for the increased loading.

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Figure 20. Photo. Crowchild Trail Bridge in Calgary

Hurricane Deck Bridge Replacement, MO

The Hurricane Deck Bridge in Camden County, MO was investigated for potential foundation reuse (Axtell et al. 2014). Although the replacement bridge did not eventually reuse the foundation, reuse was considered during the planning phase, and significant testing was performed on the existing piers and abutments. This case study is included to show that the foundation and substructure of this bridge had significant functional value for another potential service life, and to demonstrate other factors that may make foundation reuse infeasible.

During the design phase, both rehabilitation and replacement alternatives were considered. MoDOT performed preliminary investigations to determine the feasibility of reusing the bridge piers. The proposed solution was to build a temporary foundation adjacent to the existing piers to hold the old deck while the first half of the new deck was constructed on top of the refurbished piers. Traffic would be moved to the new deck while the old deck and temporary foundation were removed, and the second half of the new deck was constructed. However, the winning bid utilized a new alignment that reduced the span lengths and increased the number of piers from 4 to 7.

The investigation consisted of reviewing available foundation records and evaluating the existing caissons with borings performed from the deck (at least two per caisson) of the existing bridge, as shown in figure 21. Cross Sonic Logging (CSL) was performed between the two borings, but did not provide meaningful results, believed to be due to the large distance between holes. Other testing performed during coring consisted of evaluating core recovery, laboratory testing, and use of an acoustic televiewer. The laboratory testing included petrographic analysis on removed core samples. The petrographic analysis did not identify any microcracking or honeycombing and found the concrete to well-consolidated and well-mixed (Axtell et al. 2014). The core samples showed good recovery (90 percent), although there was a large range to the rock quality designation (RQD), and the average RQD was 60 percent. The existing caissons were evaluated against current design codes for the potential new superstructure loads. From this testing, the existing foundations were deemed to be candidates for reuse. According to Axtell et al. (2014), "This conclusion was valid even considering the substantially more stringent design codes in effect today relative to those that may have existed in the 1920s when the original bridge was designed."

The replacement alternative for this project was expected to be significantly more expensive than the retrofit cost. During the bidding process, a design-build contractor proposed a replacement bridge design utilizing the existing right of way of the bridge. The selected bid for a steel girder bridge replacement was 1 percent cheaper than the reuse alternative using a prefabricated delta truss. However, the replacement bridge is traditional steel girder bridge which is not as elegant as the retrofitted bridge using delta truss. Notably, the design-build contractor took the significant risk of demolishing the existing bridge in the proximity of the new bridge. Furthermore, this case example demonstrates that existing bridge foundations can still be suitable for reuse after 90 years, even though that alternative was not selected.

Figure 21. Photo. Drilling from bridge deck on Hurricane Deck Bridge

CHAPTER 2. REUSE DECISION MODEL

INTRODUCTION

The primary objective of this chapter is to identify both the technical and non-technical factors that affect the decision-making process leading to foundation reuse. Despite the potential for significant savings with respect to both the total cost and the construction time, there are risks associated with foundation reuse, including uncertain evaluation and rehabilitation costs that can weigh on these potential benefits. Defining a comprehensive process and procedure for assessing the risks and feasibility of reuse projects allows for informed decision-making on the potential benefits of foundation reuse.

A foundation reuse decision model is a systematic process that enables bridge owners and agencies make an optimal decision. Picking the safest, most economical option requires comprehensive evaluation of various aspects of each alternative. Since the required information may be either impossible or expensive to collect during the initial and preliminary stages of decision-making procedure, implementation of a hierarchical process can save time and money. In the beginning of the decision-making process, a comparison between proposed alternatives is performed by using minimum level of information or readily available data. Subsequently, more detailed data is acquired for the remaining options and this procedure is continued until the optimal option is identified.

Various research has previously been conducted on the decision-making process for the reuse of building foundations. The RuFUS best practice handbook (Butcher et al. 2006) provides flowcharts for managing technical and legal issues associated with reuse. Important criteria for assessing the risks and benefits of foundation reuse are identified as construction costs, whole life costs, and environmental impact. Chapman et al. (2007) provides a decision-making process consisting of desk study, assessing current conditions, monitoring to verify performance, field investigations, foundation performance, and assigning a new working/design capacity. Straus et al. (2007) detailed the Sustainable Project Appraisal Routine (SPeAR®), which created diagrams that qualitatively identify how feasible are reuse for building foundations based on location. Eight drivers for reuse were considered, and the significance of each driver was qualitatively placed on the bull's eye shaped diagram shown in figure 2. The locations with the most drivers significantly influencing decision-making were considered the most well-suited to foundation reuse. Laefer (2011) modified the SPeAR approach to quantify reuse potential by assigning a quantitative value (from 1 to 6) to the qualitative reuse driver categories. Laefer and Farrell (2014) proposed a hybrid reuse evaluation approach by combining the modified SPeAR® diagram method with the modified RuFUS flowchart. This approach allowed considering both socioeconomic and technical factors in the reuse decision process.

These publications outline the drivers and decision-making process for the reuse of building foundations. Bridge foundations will be subjected to many of the same considerations as building foundations, although reuse of a bridge foundation entails consideration of additional constraints, challenges, and potential benefits. The reuse decision-making process explored in this chapter accounts for the issues specific to bridge foundations to enable optimal selection of reuse/replacement alternatives. The major portions of the decision-making process are:

- Desk study (in-situ geometry, compatibility between new and existing footprints, past performance, identification of alternatives)
- Integrity, durability, and capacity assessment of the existing foundation
- Analysis of scour and hazard vulnerability
- Assessing constructability of proposed alternatives
- Environmental impacts assessment
- Risk analysis of proposed alternatives
- Life-cycle cost analysis of available options.
- Alternative Selection

The flowchart in figure 22 shows this decision-making process:

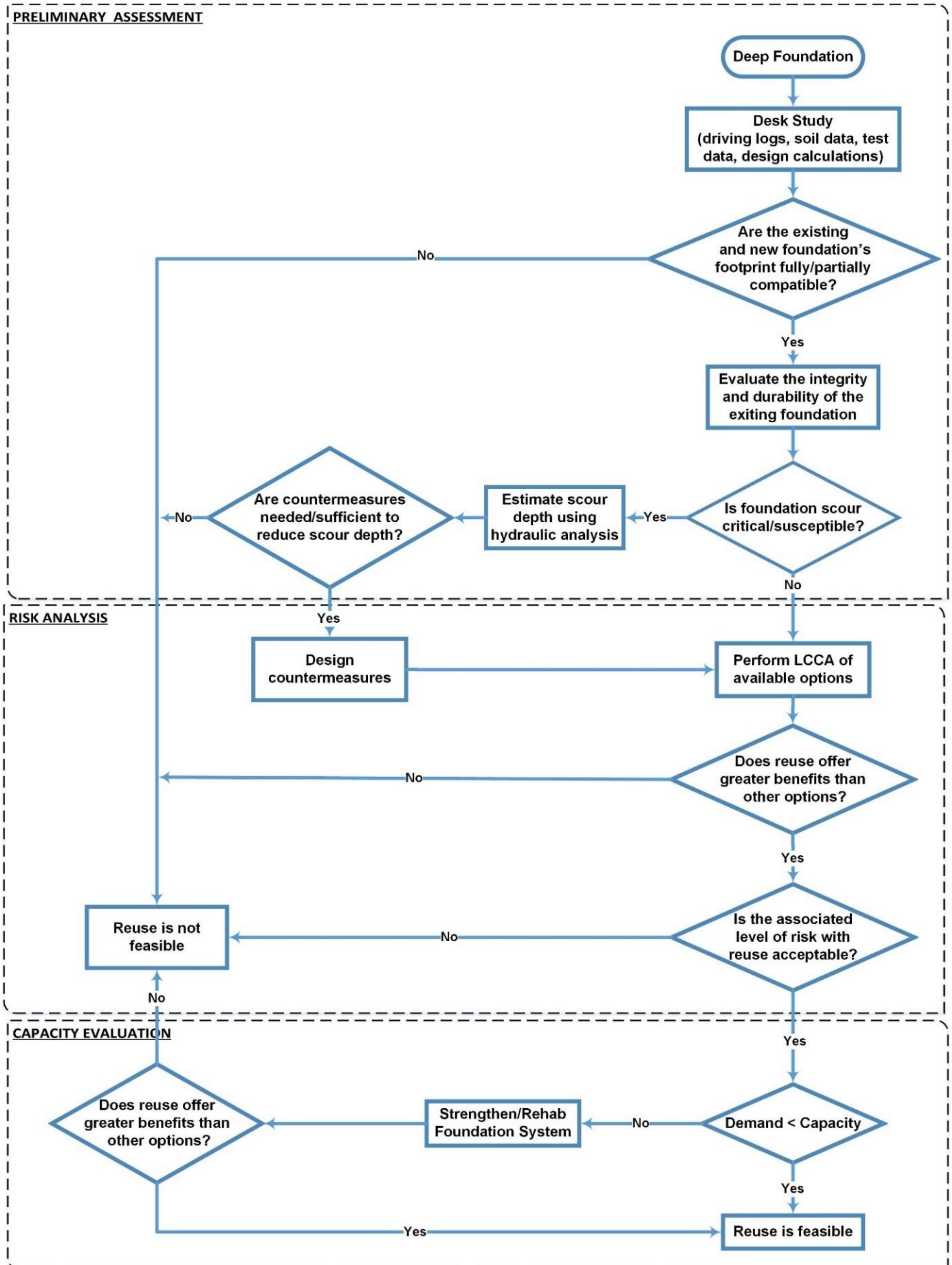


Figure 22. Chart. Decision Making Procedure for Reuse of Bridge Foundations

DESK STUDY

Primary Details

NCHRP Synthesis 505 (Boeckmann and Loehr 2017) defined the "primary details" of an existing foundation as the type of foundation (shallow versus deep), dimensions of the foundation elements, location of the foundation elements, and the depth of the foundation. These details are crucial to determining if the current foundation is compatible with new superstructure design alternatives.

Geometric Compatibility

The proposed alignment of the roadway can be compared with as-built plans of the existing foundation to determine how compatible the existing and proposed substructures are. The footprints of the existing foundations can be fully or partially compatible with the proposed foundation. If an existing foundation does not fully align with proposed piers or abutments, various options exist for overcoming these geometric compatibility conflicts. An example includes removal of some of the existing piles and using a portal cap between the remaining old piles and new piles.

The amount of geometric compatibility between the new and existing foundations and the sufficiency of the existing foundation capacity is important when determining which of the four options (figure 1) are suitable for bridge reconstruction. The level of geometric compatibility can be defined as a percentage of piles compatible for deep foundations. The percentage of geometric compatibility for shallow foundations can be defined based on the common footprint area. Partial reuse of shallow foundations (isolated footing, continuous strip footing, combined footing and mat foundations) may require special considerations. The percentage of geometric compatibility can vary from zero (meaning no compatibility) to 100 percent (meaning full compatibility). Reuse of an even small percentage of piles is theoretically possible but may not be economically feasible. An acceptable value of minimum percentage of geometric compatibility varies by project and potential economic benefits.

Past Performance

Preliminary decision-making is greatly aided by determining how well the existing foundation has maintained the loads and environmental conditions imposed during the initial service life. Signs of existing damage or evidence of poor performance or/records of structural repairs, are indications that the present structure may not be performing suitably to warrant additional service life. Bridges displaying a history of performance issues often will incur additional investigation, repair and strengthening costs.

Alternative Identification

Initially identifying multiple viable reuse and/or reconstruction alternatives can be advantageous during alternative selection. These alternatives can be considered by the owner agency as part of a selection process or chosen by the contractor as part of a design-build or construction-manager general contractor (CMGC) type of bid process. It often makes sense for the owner agency to narrow the potential options to a set of acceptable or feasible alternatives, while allowing contractors to select and design other details. The following decision-making process can be applied to these identified alternatives for selection of the optimal alternative.

INTEGRITY, CAPACITY, AND DURABILITY ASSESSMENTS

When identifying a foundation as a candidate for reuse, there are three major concerns which are typically addressed: integrity, capacity, and durability, which are addressed in Chapters 4, 5, and 6 of this manual, respectively. Each of these areas of assessment is aided by local experience and familiarity with the reuse of foundations. These assessments begin with fact-finding of the existing structure, progress to identifying areas where knowledge is lacking, then take the appropriate steps toward eliminating these gaps in the knowledge, as discussed in Chapter 3 on "Data Collection and Assessment Procedure." By comparing several potential reuse and replacement alternatives against each other, the most suitable option can be chosen.

EXTREME HAZARD VULNERABILITY

NCHRP 489 (Ghosn et al. 2003) considered four types of extreme events that could impact a bridge: earthquakes, winds, scour, and ship collisions. Other extreme hazards that may impact a bridge include but aren't limited to: vehicular impacts, exposure of bridge components to fire, ice/debris flows, and blasts. These hazards are often infrequent but have the potential to cause extreme structural demands. In recent decades, the analysis of these hazards has seen considerable research and advancement in analysis techniques and standards. Existing foundations may have been built to obsolete codes and/or never subjected to modern standards of care during their construction and initial service life. When being considered for a new 75-year design life, evaluating the resilience of the bridge to future hazards can present unique design challenges. Existing bridges with uncertainties related to the initial design may require additional evaluation to completely analyze geo/hydraulic hazards and other extreme events. Bridges with insufficient capacity or substandard design details may require retrofit for them to be suitable for reuse.

Scour is the primary cause of bridge failures in the United States. (Melville and Coleman 2000; Briaud et al. 2001; Briaud et al. 2005). Hence, the vulnerability of existing bridge foundations to scour is a significant risk to be considered during foundation reuse when applicable. In NCHRP Synthesis 505 (Boeckmann and Loehr 2017), participating DOTs and other agencies were asked "will foundations be reused when scour countermeasures are required." The pie chart in figure 23 shows the response of participants. Among 48 agencies responding to this question, 21 (44 percent) answered "yes", 19 (40 percent) answered "no", and 8 (17 percent) answered "not sure".

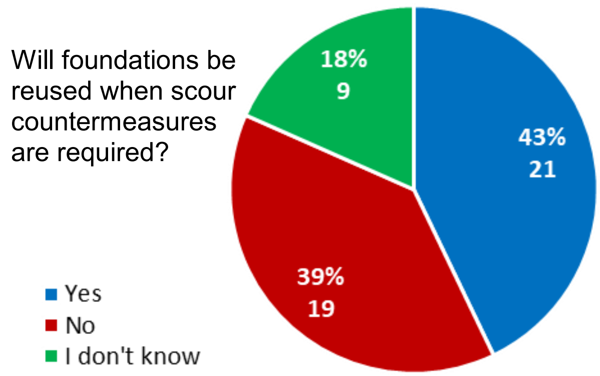


Figure 23. Chart. Survey on reuse of bridge foundations when scour countermeasures are required NCHRP Synthesis 505

CONSTRUCTABILITY

Constructability broadly refers to the ability of a design to be transferred from drawings to reality. A constructible design is one that minimizes expenses, design changes, errors, and delays during construction. NCHRP Synthesis 505 (Boeckmann and Loehr 2017) shows that 46 percent of the DOTs reused foundations because of constructability reasons. Scrutiny of available alternatives from a constructability perspective is an important component of the decision-making process.

The constructability review of projects can be conducted in two stages: pre-design stage and design stage (Idaho DOT 2011). Construction procedures for Option 1 are almost the same as for new foundation construction. The only issues that may require special consideration relates to clearance from the existing bridge, disturbance of the existing bridge if it is left in-service during construction, and changes to the roadway alignment. Interference issues include existing battered piles that limit the room for new foundation elements, vibration sensitivity of the new structure, excavation considerations, scour issues, demolition of the existing foundation above the mudline, and space required for construction. Option 2 alternatives often require construction activities on or underneath active roadways, requiring consideration of these impacts. Interference and impacts on the existing substructure elements are also important considerations, as well as how the new superstructure will be placed on the original alignment. Option 3 alternatives require the least construction, so constructability issues will revolve around deck placement. A major consideration for Option 4 alternatives is the impact strengthening will have on the structure if it is kept in use during construction.

ENVIRONMENTAL IMPACTS ASSESSMENT (EIA)

Environmental Impact Assessment (EIA) is the process by which the anticipated effects on the environment from a project are measured. Foundation reuse is likely to have significantly lower environmental impacts than alternatives from Options 1 and 2 through savings of material and labor. A number of Federal and State statutes provide guidance and requirements of the EIA. For instance, the New York City Environmental Quality Review Manual (NYCEQR 2014) has provided a checklist for the initial identification of the issues to be addressed in the environmental impact statement.

Environmental Impacts Assessment Areas

Based on the NYCEQR (2014), environmental impact assessments may require initial identification of issues in the following technical areas:

- Land use, zoning, and public policy
- Socioeconomic conditions
- Community facilities and services
- Open space
- Shadows
- Historic and cultural resources
- Urban design and visual resources
- Natural resources
- Hazardous materials
- Water and sewer infrastructure
- Solid waste and sanitation services
- Energy
- Transportation
- Air quality
- Greenhouse gas emissions
- Noise
- Public health
- Neighborhood character

Not all the aforementioned areas would be applicable to any specific construction project. These considerations are most applicable to foundations being considered for reuse, but for any specific foundation reuse project, other considerations may apply.

Land Use, Zoning, and Public Policy

The land use analysis characterizes use and development trends in the area that may be affected by a proposed project. The compatibility of the project with local land use, zoning, and public policy is investigated. Similarly, the analysis considers the project's compliance with, and effect on, the area's zoning and other applicable public policies. Of the available foundation options, only Option 1 requires significant land acquisition.

Natural Resources

A natural resource is defined as any aquatic or terrestrial area capable of providing suitable habitat to sustain the life processes of plants, wildlife, and other organisms. Areas that function in support of the ecological systems that maintain environmental stability are also considered natural resources. Options 1 and 2 will generally have the greatest impact on natural resources, while Option 3 will typically have the lowest impact on natural resources. Option 4 alternatives will typically have a lesser impact on natural resources in comparison to Options 1 and 2 alternatives.

Solid Waste

A solid waste assessment determines whether a project has the potential to cause a substantial increase in solid waste production that may overburden available waste management capacity or otherwise be inconsistent with State policies. Any demolition performed as part of bridge replacement will lead to solid waste disposal.

Energy Consumption

The energy consumption analysis considers all the energy sources typically used in a project’s operation, including electricity, fossil fuels (oil, coal, gas, etc.) and other required sources of energy. The sources of energy consumption in construction projects can be categorized in three parts: material manufacturing, transportation, and construction (Hong et al. 2014). Energy consumption in transportation includes transportation of materials as well as increased consumption because of impacts on mobility. Option 1 consumes energy for demolition of the existing foundation, material manufacturing, transportation and construction. Option 2 consumes energy for demolition of the entire existing foundation, material manufacturing, transportation and construction. Option 3 consumes energy for in-field investigation and Option 4 consumes energy for in-field investigation, material manufacturing, transportation, and construction.

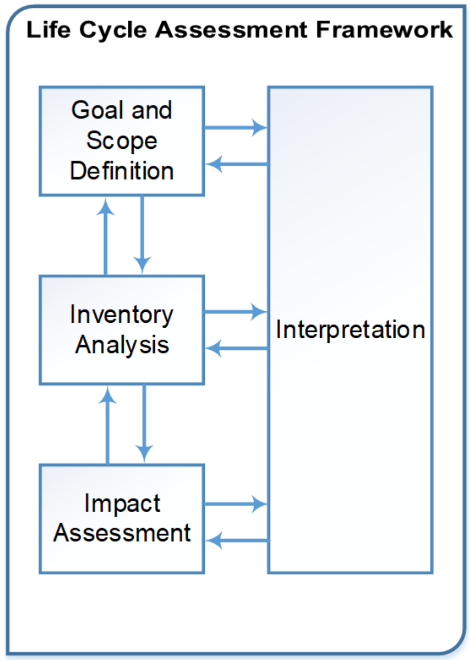
Air Quality

Generally, an air quality assessment determines a proposed project's effects on ambient air quality. Proposed projects may impact air quality during operation and/or construction. In terms of air pollution, the greenhouse gas (GHG) emissions during a project are considered to evaluate the project’s effect on ambient air quality. As a general statement, for all 4 available Options, alternatives with lower construction and material usage will have lower impacts on ambient air quality. Often, the lowest impact case will be for complete reuse of the existing foundation, when possible.

The literature review of the environmental impact assessment for civil projects shows that the quantitative analysis and evaluation of all abovementioned items is not feasible. In addition, each State/City has its own specification and/or guideline for assessment of environmental effects and consequences of the construction projects.

Life Cost Assessment (LCA)

Life Cost Assessment is a methodology that evaluates the environmental impacts of a product or a system during its life. The LCA can help decision-makers select the process that results in the least impact to the environment. An LCA allows decision makers to study the entire optional system, rather than focusing on individual processes. For foundation reuse, LCA may be useful as a systematic tool for environmental impact assessment of available foundation replacement/reuse alternatives. As shown in figure 24, ISO 14040 standards divide LCA methodology in four fundamental and complementary sections: goal and scope of the study, life cycle inventory (LCI), life cycle impact assessment (LCIA), and interpretation.



Original photo ©2006 ISO (See Acknowledgments section)

Figure 24. Illustration. Four parts of life cycle assessment methodology

RISK MANAGEMENT IN FOUNDATION REUSE

Risk management has been defined as the process of identifying and assessing risk and applying methods to reduce it to an acceptable level (Tohidi 2011, Lee et al 2009). These risks can be managed through a risk management process (RMP) that evaluates various alternatives with differing risks. Smith et al. (2014) and Gajewska and Ropel (2011) describe a four-step risk management process (RMP) that can be used to control the risks of a construction project. The steps of the RMP consist of:

- Identification of risks,
- Assessment of identified risks,
- Development of risk response, and
- Monitoring of the identified risks.

A possible RMP is suggested herein that follows these steps.

Identification of Potential Risks

Risk can be defined as an uncertain event or condition that can impact the outcomes of a project (PMI 2013). Risk is the product of both the probability of a condition occurring and the consequence, positive or negative, this condition has on a project. Normal bridge construction presents bridge owners and stakeholders with various risks that can negatively impact safety, construction costs/impacts, and life cycle costs. Rehabilitation, reuse, or reconstruction projects (from any of the four options) present additional risks that can impact the above-mentioned risk areas. These additional risks primarily originate from: the assessment of the condition and capacity of the existing structure; the impacts of construction on or near an active right of way (ROW), and the impacts of construction activities on the surrounding environment.

The risks for a foundation reuse project can be identified using the following evaluation process:

- Recognition of primary sources of risks associated with reuse (local knowledge and experience driven)
- Desk study of available information and knowledge on the existing bridge
- Evaluation of old foundation
    - Material strengths
    - Element integrity and durability (spalling, cracking, corrosion, decay, deterioration)
    - Effects of extreme loads and conditions (scour/hydraulic, seismic, wind, etc.)

The potential impacts identified above will be driven by individual risk factors that are specific to each location and bridge type. Identification of these risk factors will depend heavily on experience. While risks cannot be eliminated, their impacts can be controlled and minimized, if they have been identified.

Sources of Risk

New construction faces various risks during and after construction that can weigh on project selection and outcomes. Cost overruns, right of way (ROW) acquisition issues, scheduling uncertainty, extreme weather during construction, accidents, and releases of pollution can occur on all construction sites. Since they are ubiquitous in construction, they are generally well understood and managed by agencies and stakeholders. Safety risks stemming from design and material uncertainty, construction flaws, and extreme events are handled by modern code standards and QA/QC practices that have been developed to avoid or mitigate these risks. Similarly, the future life cycle of new bridge is well understood, and code provisions exist to mitigate these risks.

Since Option 1 alternatives involve installation of completely new foundation elements, there are few risks associated with the condition of the existing bridge. The risks that impact Option 1 alternatives primarily revolve around the impacts of the new construction near the existing construction. These activities can cause accidental damage to the existing foundations, which are often in-use. If damage were to occur to the existing foundation, there would be potential impacts such as, safety implications for workers and users of the existing bridge, mobility impacts from lost usability of the existing bridge, cost impacts, and scheduling impacts. In addition, Option 1 bridges generally require some form of ROW realignment that can prevent risks to user mobility if problems arise during the realignment.

Option 2 bridges also involve installation of completely new foundation elements, eliminating the impact of the condition of the existing bridge. Since the existing construction will be demolished prior to construction, the potential impacts on user safety and mobility during construction are avoided. The demolition of the existing foundation may impact the surrounding environment and could result in releases of debris and pollution without adequate control. Since the ROW will be closed during construction and reconstruction, scheduling delays can have outsized impacts on user mobility, as the ROW is unusable during the entirety of construction.

Option 3 alternatives are subjected to additional risks due to the use of existing components that may be of uncertain initial quality, condition, or design. Existing elements may have unknown properties, including material strength, reinforcement layout, or geometry. Design plans of the existing foundation may not match the as-built condition or may be non-existent. The existing system may lack key test data that would normally be obtained for new construction, including testing of driven piles or concrete specimens. The existing elements may have degraded or become damaged during their initial lifespan. Infiltration of chlorides can impact the remaining lifespan of existing concrete elements, steel elements may have begun to experience corrosion, and timber element may have begun to experience decay.

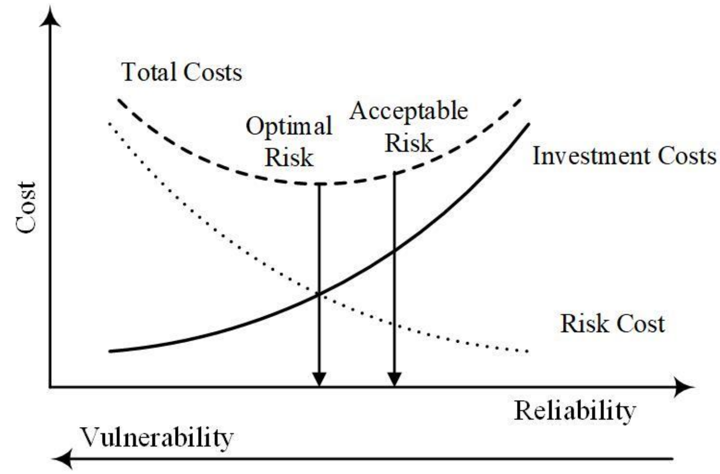
Option 4 bridges are subjected to many of the same risks as Option 3 bridges due to the use of existing components. The strengthening process introduces additional scheduling and design risks during the strengthening phase that can impact the overall suitability of the design alternative. The use of strengthening can also mitigate some of the risks associated with reusing an existing design. Option 4 bridges may also be subjected to additional risks that arise during the strengthening process, especially when the strengthening is performed on in-service bridges.

Assessment of Impacts from Identified Risks

The impact of risks arising from foundation reuse can be described in terms of financial consequences. Inclusion of the risk costs may show that reuse has higher costs than a new foundation, even if it has a lower estimated cost. Other than direct costs, the other impacts identified above (scheduling, environmental quality, safety, impacts on surroundings/aesthetics, user mobility, and historic resources) are referred to as indirect costs imposed on local businesses and the community at large. The costs originate from the negative effects of bridge closure (change in bus schedules, heavy good vehicles (HGV) routed through residential roads, longer routes for emergency vehicles, and economic impact on local industry and businesses). Although these costs cannot be quantified easily, they have negative effect on the community.

Acceptable Risk

The amount of risk an agency will accept for a project will be influenced by that agency's tolerance to risk, the optimal financial solution, and local sentiment. Figure 25 shows the concept of cost-based risk selection, using an idealized price/risk curve. The dotted, solid and dashed curves represent the risk, investment, and total costs, respectively. As a generality, mitigating, avoiding, or transferring risk will lead to higher investment costs and lower risk costs. Retaining additional risk typically raises risk cost but allows for lower investment costs. There is an "optimal risk" point where the total costs (the sum of investment and risk costs) are minimized. If an agency is too tolerable to risk, the total costs will increase, even if the initial investment costs are lower. When agencies are overly hesitant to take on risk, the total costs for the project will increase, but the potential for future risk costs will be lowered. To a certain effect, this may be desirable, although difficult to justify from a purely economic perspective. Figure 25 shows an illustration of these concepts, with the y axis representing increasing costs and the x axis representing increasing reliability.



Original photo: ©2017 husdal.com (see acknowledgments)

Figure 25. Illustration. Optimal versus acceptable risks

The amount of acceptable risk can be determined from the definition of objectives, available alternatives, consequences of the risks, and likelihood of occurrence (Ayyub et al. 1999). In terms of foundation reuse, the risk criteria outline in the LRFD Bridge Specifications (AASHTO 2014) is generally followed. AASHTO (2014) often assumes a target reliability index, β, of 3.5 to derive resistance factors for structural components. A reliability index of 3.5 corresponds to an approximate probability of failure, P_f, of 0.0002. Past geotechnical design practice has resulted in an effective β of 3.0, (approximate probability of a failure of 0.001) for foundations in general, and an approximate reliability index of 2.3 for pile groups (approximate probability of failure of 0.01). Hence, resistance factors recommended in LRFD Specifications (AASHTO 2014) are based on these target reliability indices.

Risk Response

Once the impacts from the identified risks are understood, strategies for reduction or mitigation of the identified risks can be considered. Potts and Ankrah (2008) provides four potential responses to identified risks: avoidance, reduction (mitigation), transfer, and retention.

Avoidance

Avoidance may be an appropriate strategy when the consequences of the impact are unacceptable (such as loss of life) or when mitigation costs would be unacceptably high. Avoidance is often the simplest form of handling risk, as it requires no further analysis beyond appropriate alternative selection. The types of impacts that are typically avoided during foundation reuse projects include cost impacts, environmental impacts, safety issues, and loss of historical resources.

Situations where direct cost risks can be avoided include dismissing alternatives with significant cost uncertainties. For example, an alternative may be under consideration that would require extensive analysis of the existing substructure to determine if reuse is feasible and what strengthening would be required. Prior to conducting this investigation, there may be significant uncertainty behind the amount of investigation that is required and how much strengthening would be needed. If the investigation determines reuse is not feasible, those investigation costs would be lost. While in many cases it would worthwhile to perform this investigation, if a total reconstruction alternative is sufficiently attractive, avoiding the cost risks associated with assessment and constructing a new foundation without assessing the existing one may be desirable.

Environmental impact risks can stem from investigation or construction activities that can negatively impact the surrounding environment (runoff, soil entry into water, contamination). Generally, mitigation procedures are followed to limit the potential impacts and their likelihood of the risks associated with a project. Choosing an alternative that does not present these risks may be possible, eliminating the need to mitigate the risk during construction or to transfer the risk to the contractor. Similarly, choosing alternatives that do not present safety risks (during or after construction) and that do not risk historical resources would be examples of risk avoidance.

Mitigation

Risks can be mitigated by performing investigations, making design choices, selecting alternatives, or performing monitoring that reduces the consequence or likelihood of a potential impact to an acceptable level. For example, there may be safety risks due to incomplete knowledge of the foundation depth that can be mitigated with parallel seismic testing to determine the foundation depth. Similarly, there may be durability concerns that prompt life cycle cost risks that can impact the costs of a project. Retrofits like wrapping or cathodic protection can be employed to mitigate these cost risk factors. Soil improvement techniques to avoid liquefaction or slope stability problems are other examples of risk mitigation strategies. Risk mitigation typically carries an immediate cost related to the mitigation which could make a selected alternative not feasible.

Transfer

In some cases, it may be best to transfer some of the project risks to another party. Project delivery methods like design-build (DB), alternative technical concepts (ATC), and Construction Manager/General Contractor (CMGC) can be employed to help limit the amount of risk to which the owner agency is exposed. The delivery methods can be particularly advantageous in accelerated construction, historic preservation, and other scenarios where foundation reuse is often considered. Delivering projects in this manner allows agencies to perform initial investigations and then solicit alternatives from contractors and designers which include the costs for managing the risks of that alternative. For example, during the Hurricane Deck Bridge reuse study, the winning contractor submitted an ATC bid that was 1 percent cheaper than the baseline case. The alternative concept did not reuse the bridge but built a new one alongside of the existing bridge where the temporary piers were proposed. In this instance, the risk of damaging the existing foundation while it was still in-service was transferred to the contractor.

Another example of risk transfer includes pricing delivery dates into the contract. The contractor may typically incur some form of penalties for delays beyond the delivery date. The contractor may also be rewarded for finishing the project before the delivery date. This can have the dual benefit of motivating the contract to deliver the project on time or earlier, while recouping some losses related to indirect impacts from bridge closure. For bridges where reuse alternatives are being driven by construction scheduling and user impacts, this can further incentivize the contractor to deliver the project within defined time parameters.

Retention

When a risk cannot be transferred or avoided, the risk is considered "retained." Retained risk refers to the acceptance of losses related to an unresolved impact (Potts and Ankrah 2008). Risk retention is not an option for critical items like life safety or capacity but can be considered for other negative impacts like limited service life, loss of historic character, traffic impacts, or future required maintenance, when other solutions are uneconomical (Thomas 2009).

Monitoring of Risks

Risks can be monitored during a project to reduce their potential impact. Monitoring of risks includes a large amount of potential actions, from ensuring milestones are met, monitoring negative impacts like settlement of adjacent structures or pollutants leaving a site, and reviewing designs submitted by a contractor.

LIFE CYCLE COST ANALYSIS (LCCA)

Life Cycle Cost Analysis (LCCA) is a methodology for comparing the costs of project alternatives over the entirety of the project lifespan. Between 1995 and 1998, LCCA was required to be performed on all transportation projects with more than $25 million federal contribution (Walls and Smith 1998). After that time, LCCA was no longer required on large projects, but the use of LCCA was encouraged by the FHWA. Walls and Smith (1998) outline an LCCA methodology for estimating life cycle costs of pavement design projects. The USDOT Office of Asset Management published a primer in 2002 (FHWA 2002) detailing the LCCA methodology and discussing the implementation to that point on the state level. For projects where foundation reuse is being considered, LCCA can be a highly useful tool, as it allows agencies and stakeholders to evaluate the lifespan costs of various alternatives, even when they require future action. For instance, an alternative may present itself with lower initial costs, but may require larger future maintenance costs. The LCCA methodology also allows simultaneous inclusion of direct costs (actual expenditures by the agency) and user costs (impacts of construction, maintenance, closure, etc. on bridge users).

The LCCA process steps for reuse projects can be listed as below (FHWA 2002):

1. Establishing alternatives
2. Determining analysis period
3. Cost estimation (agency and user)
4. Life-cycle costs computation
5. Comparison of alternatives

Establishing Alternatives

LCCA is performed using a set of identified alternatives, in this case taken from one of the four options listed in figure 1. By estimating the future maintenance, repairs, and bridge closures for each alternative, the ultimate cost of various options can be compared. For example, if a reuse alternative is projected to have higher repair costs than a replacement alternative, or if one alternative is expected to require a midlife overhaul, the total cost of these options is not well represented by the initial construction cost.

Determining Analysis Period (Activity Timing)

LCCA requires that all alternatives be compared using a common analysis period. The analysis period is the timeframe over which future costs will be considered and included in the lifecycle cost. For Option 1 or 2 alternatives, agencies typically have a defined minimum service life (often 75 years), defining the analysis period from the start. When considering reuse alternatives (Options 3 or 4), these considerations may become more complex, as reused components could potentially have diminished service lives or require mid-life rehabilitation. Extending the analysis period beyond the initial service life for each alternative and including at least 1 major repair or replacement in the analysis period allows for comparison of these alternatives. Projecting the remaining service life for Option 3 and 4 projects can add uncertainty to the LCCA, due to uncertainty in the model used to estimate remaining service life. For Option 1 and 2 bridges, it may be difficult to estimate how much or if the substructure will have usable value at the end of its 75-year service life. The further into the future the costs are projected, the more uncertain will the analysis become due to less certain discount rates, future costs, and lifespan projections. In many cases, the analysis will be sensitive to the length of analysis period used and viewing multiple analysis periods can allow for more informed decision-making.

Figure 26, adapted from Walls and Smith (1998), depicts a sample lifespan and condition comparison. This sample lifespan includes mid-life rehabilitation but does not include the entire lifespan or end-of-life costs.

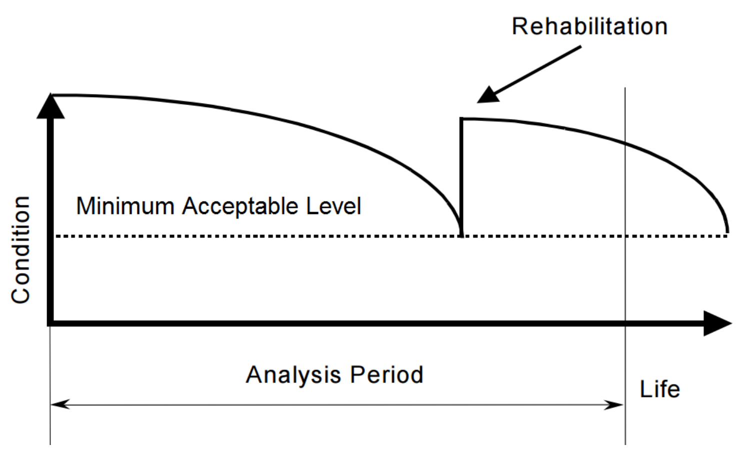


Figure 26. Illustration. Lifetime of a design option

Costs Estimation

There are two types of costs related to bridge serviceability: agency costs and user costs. Agency costs are driven by the costs of performing the requisite investigation, designing the alternative (perhaps at multiple levels of design), and constructing the chosen alternative. To estimate the costs of different activities, a good cost-accounting system and local experience are essential. Performing early design on multiple alternatives will cause some initial direct costs, though these costs are often minimal in comparison to the potential for reduced costs in other stages of the project. User costs include accounting for any loss of the structure during construction, future maintenance/rehabilitation, potential future load posting, and the impact of any future deficiencies of the alternatives. These loss of service time or functional deficiencies may cause higher vehicle-operating costs resulting from detours, additional travel time, and loss of functionality.

Agency Costs

Investigation Costs

The extent of investigation required depends on the quality and quantity of information available during the desk study. Reuse alternatives will typically require more extensive investigation of the geometry and integrity of the existing foundation than those for Option 1 or 2 alternatives. Commonly, investigations are divided into two broad categories: an initial assessment and an in-depth assessment. The initial assessment can be used to determine which parameters need to be further investigated. Option 1 bridges will often incur additional costs related to the geotechnical exploration, as Option 2, 3, and 4 bridges will typically have existing geotechnical data and will only require supplemental geotechnical data.

Design Costs

Design costs will exist for any alternative and are those incurred during planning and design of a project. Design costs are often lower for a project involving new foundations than for a foundation reuse project. This is attributable to additional design and analysis work required to characterize the in-situ element over a routine design. Option 4 alternatives may require transfer structures that compensate for incompatible behavior of the elements or the foundation, which can increase both design and construction costs.

Construction Costs

The construction costs associated with complete replacement (Options 1 and 2) are often higher than reuse, but easier to quantify. Option 1 requires land and right of way acquisition that can drastically increase costs. Option 2 requires demolition of the existing foundation and material removal. Options 3 and 4 are often able to derive savings from using the functional value of the existing components, but uncertainties related to repair and retrofit may increase construction cost risks. When these construction cost risks are priced into the contractor's bid (i.e. transferred to the contractor), the agency cost to reuse will may increase.

Indirect Costs

Costs not directly related to construction activities are considered user, or indirect costs, and can be grouped into project overhead and general office overhead. These costs include insurance costs, facility costs, and all other costs that are not related to a specific project activity.

User Costs

User costs, shown in Equation (1) as C_{user}, can be considered as the summation of the additional vehicle operating cost *VOC*, travel time cost *TTC,* and accident cost *AC* that would result from closure of the bridge (Imhof 2004).

$$C_{user} = VOC + TTC + AC \tag{1}$$

where,

$$VOC = AADT \times VOC_u \times L_d \tag{2}$$

$$TTC = \frac{g_{day} L_d n_p AADT}{V} \tag{3}$$

$$AC = AR.L_d.ICAF \tag{4}$$

Where $AADT$ is the annual average daily traffic, VOC_u is the unit vehicle operating costs ($ per mile per vehicle), L_d is the additional length of the detour route, g_{day} is the per capita daily GDP for population using the bridge, n_p is the average number of people per vehicle, V is the average velocity of vehicles, AR is the accident rate on original route (fatalities per mile per day), and ICAF are the implied costs of averting a fatality.

The above equations provide the daily user costs. The total user costs are the daily costs multiplied by the estimated time of traffic disruption. Obtaining the daily costs is useful for determining contractor penalties and incentives, if used, for limiting bridge closure time. The calculation of user costs, however, is highly dependent on the assumptions of traffic volume and the delays associated with bridge construction (Imhof 2004). While these costs are diffuse and cannot be exactly measured, they give a numerical, dollar-based approximation of the amount of disruption expected from bridge construction work.

Life-Cycle Costs

After the alternatives, their associated agency and user costs, and the analysis period have been defined, the total cost of each option may be calculated and compared. The costs that have been estimated are typically converted to their value at a common point of time for all identified alternatives. Conversion of costs to the present value (PV) is suggested by the FHWA. The formula to discount future costs to present value is:

$$\text{Present Value} = \text{Future Value} \times \frac{1}{(1+r)^n} \tag{5}$$

where r = discount rate and n = number of time periods until cost is incurred. The units of r and n need to be consistent, and years are commonly chosen. Real discount rates used in life-cycle cost analyses typically range from three to five percent, representing the prevailing rate of interest on borrowed funds minus expected inflation. The term $1/(1+r)^n$ in equation (5) is known as the discount factor and is almost always less than or equal to one.

There are two approaches for computation of costs: deterministic and probabilistic. The deterministic approach assigns each LCCA input variable a fixed, discrete value at a discrete instance of time, which is most likely to occur for each input parameter. A deterministic method is straightforward, but it does not consider uncertainties associated with future costs. A probabilistic approach defines the input variables with several possibilities, each with its own costs and probability of occurrence. More advanced analysis can model parameters as random variable with probability density functions. Probabilistic models provide a way to account for future uncertainty but require understanding of the likelihood of an event or the distribution of variables used.

Total Costs

The total costs of the project are then the summation of the agency costs C_{agency} (initial design, construction and periodic maintenance and rehabilitation activities), the user costs C_{user} (vehicle operating costs, travel time costs, and crash costs) and the risk costs C_{risk}, after normalization to a specific point in time, as shown in Eq.(6):

$$C_{tot} = C_{agency} + C_{user} + C_{risk} \tag{6}$$

Comparison of the Alternatives

The results of the LCCA for replacement options are strongly influenced by the discount rate, event timing, analysis period, and event value. A sensitivity analysis is typically performed in probabilistic models to study the effect of variation in the above-mentioned variables on the results. The procedure involves changing a single input parameter over the range of its possible values, while holding all other parameters constant, and estimating a series of PVs (output values). Each PV result will reflect the effect of the input change.

ALTERNATIVE SELECTION

Selection of the preferred alternative may need to consider several of the above-mentioned decision areas, including up-front costs, life-cycle costs, sustainability, user/community impacts, cost and life cycle risks, and constructability. Alternative selection can be qualitative, where the best alternative is identified through judgement, quantitative analysis or through scoring. Often, alternative selection is heavily influenced by public feedback, which is qualitative in nature, but can be informed through assignment of a technical score. A methodology for scoring alternatives, called analytical hierarchy process (AHP) is discussed in detail below that can account for the decision-making processes listed above.

Analytical Hierarchy Process (AHP)

Analytical hierarchy process (AHP) is a tool that can be used to evaluate various construction strategies by considering quantitative and qualitative criteria (Arurkar 2005). Several comparison methods can be implemented in AHP to evaluate the relative importance of each factor in comparison with other factors using both a numerical and qualitative scale. The AHP pairwise comparison process can assign weights or priorities to criteria that are derived from these numerical or qualitative scales.

Since the decision-making in foundation reuse projects is a multi-criterion process, implementation of the AHP can be useful to include the effects from all possible factors. The reuse decision model can use AHP to quantify the relative importance of various criteria in selecting an optimum alternative. The goal is to develop a decision-making process that chooses the most cost effective and safest alternative for new foundation construction or reuse. The AHP decision-making consists of the following six steps (Salem and Miler 2006):

- STEP 1: Structure decision hierarchy
- STEP 2: Construct comparison matrices
- STEP 3: Calculate eigenvector and eigenvalues
- STEP 4: Check consistency of matrices
- STEP 5: Evaluate and compare alternatives and make decision
- STEP 6: Perform a sensitivity analysis of the model

Structure Decision Hierarchy

- Identify the objective of the decision making

First, the overall objective or goal of the process is defined. The goal of a typical bridge project is to provide a safe and economic design. Reuse projects may also have additional objectives, such as historical preservation, minimization of road closure, etc.

- Identify the proper criteria

The available options are compared and evaluated based on the criteria needed to achieve the stated goal. All motivations and drivers for foundation reuse are addressed in these criteria. Typical important criteria for reuse projects includes safety, costs, environmental impacts, and constructability.

- Identify the sub-criteria

Each criterion can then be sub-divided into additional sub-criteria. The sub-criteria are intended to be mutually exclusive and do not have common areas. For instance, assuming safety is a main criterion for foundation reuse, integrity, durability, and load capacity can be considered as its sub-criteria.

- Identify the alternatives

The alternatives that best meet the defined criteria and fulfill the stated objectives can then be identified. As previously stated, there are four reuse/reconstruction options that alternatives can be selected from. Multiple alternatives can exist for a single option, and not all options may have alternatives.

The overall objective or goal, criteria, sub-criteria, and alternatives are then arranged in descending order, as shown in Figure 27.

Construct Comparison Matrices

Comparison matrices can be constructed to determine the impact various elements in one level have on elements in the next higher level. This allows computation of the relative impacts the elements on the lowest level have on the overall objective. Each element is then evaluated against its peers in relation to its impact on achieving the objective of the parent element. The pair-wise comparisons of the elements at each level are made in terms of importance, preference, and likelihood.

The values used to compare elements are assigned from a pre-determined scale of relative importance (Saaty 1980). The ratio scale is shown in table 5 below.

Table 5. The ratio scale for pair-wise comparison (Saaty 1980)

Importance	Definition	Explanation
1	Equal Importance	Two Activities Contribute equally to the Objective
3	Weak Importance of one over another	Experience and judgment slightly favor one activity over another
5	Essential or Strong Importance	Experience and judgment strongly favor one activity over another
7	Demonstrated Importance	An activity is strongly favored, and its dominance demonstrated in practice
9	Absolute Importance	The evidence favoring one activity over another is of the highest possible order of affirmation
2,4,6,8	Intermediate Values between two Judgments	Used to facilitate compromise between slightly differing judgments

In a typical bridge foundation reuse project, the overall objective is to reuse the existing foundation in the most cost effective and safe manner with the least environmental impact. The ability of each construction alternative selected from one of the 4 options to achieve this objective can be evaluated with respect to the following criteria:

1. Safety of the whole bridge (S)
2. Cost effectiveness of the option (C)
3. Minimum impact on the environment and ecosystems (E)
4. Functionality of the whole bridge (F)

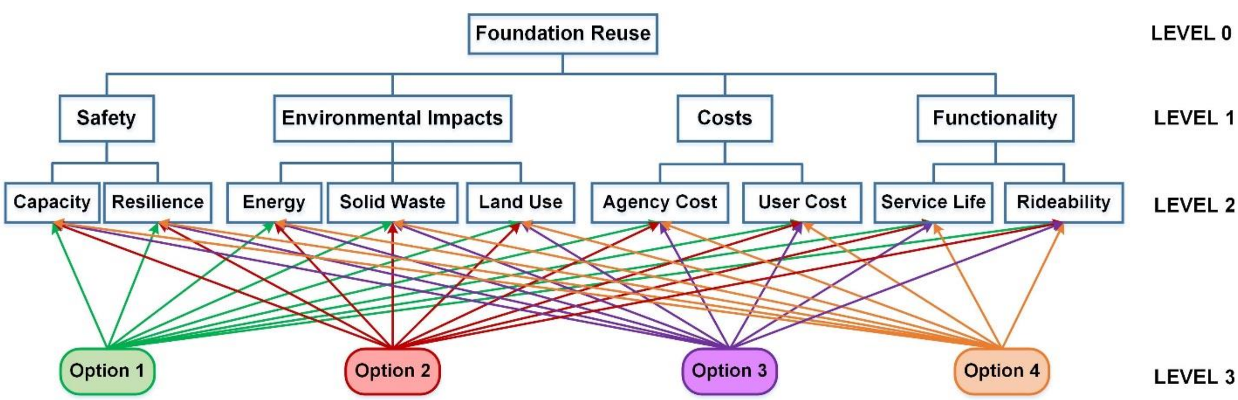


Figure 27. Chart. A sample analytical hierarchy process diagram for foundation reuse

The pair-wise comparison matrix for main factors at Level 1 is shown in Eq.(7). For example, w_S/w_C represents the pair-wise comparison of Safety criterion (S) with Cost criterion (C).

$$\begin{bmatrix} w_S / w_S & w_S / w_C & w_S / w_E & w_S / w_F \\ w_C / w_S & w_C / w_C & w_C / w_E & w_C / w_F \\ w_E / w_S & w_E / w_C & w_E / w_E & w_E / w_F \\ w_F / w_S & w_F / w_C & w_F / w_E & w_F / w_F \end{bmatrix} \tag{7}$$

Where *w* represents the weight of each criterion. The elements on the diagonal of this matrix represent the pair-wise comparison of each criterion with itself and are equal to 1.

The pair-wise comparison for these factors is likely to be unique to the project and priorities placed by decision-makers on these factors. The four main criteria in the above matrix can be further divided into sub-criteria that form their own matrices using pair-wise comparisons. The pair wise comparison matrices for sub-criteria discussed here are for demonstration of the AHP technique; each foundation reuse project is unique, and the factors discussed may or may not be used. This AHP-based model is flexible because the user can customize these matrices according to unique project requirements.

The criterion related to safety of the whole bridge can be divided into sub criteria such as (see Eq.8):

1. Structural and geotechnical load capacity (C)
2. Resilience of foundation members (R)

$$\begin{bmatrix} w_C / w_C & w_C / w_R \\ w_R / w_C & w_R / w_R \end{bmatrix} \tag{8}$$

The criterion related to the impact of construction on environment and ecosystems can be divided into sub-criteria such as (see Eq.9):

1. Land use, zoning, and public policy (L)
2. Energy Consumption (E)
3. Solid Waste (W)

$$\begin{bmatrix} w_L / w_L & w_L / w_E & w_L / w_W \\ w_E / w_L & w_E / w_E & w_E / w_W \\ w_W / w_L & w_W / w_E & w_W / w_W \end{bmatrix} \tag{9}$$

The criterion related to costs can be divided into sub-criteria such as:

1. Agency costs (A)
2. User costs (U)

A matrix of pairwise comparison costs is shown in Eq.(10):

$$\begin{bmatrix} w_A / w_A & w_A / w_U \\ w_U / w_A & w_U / w_U \end{bmatrix} \tag{10}$$

The criterion related to functionality can be divided into sub-criteria such as (see Eq.11):

1. Service life (S)
2. Rideability (R)

$$\begin{bmatrix} w_S / w_S & w_S / w_R \\ w_R / w_S & w_R / w_R \end{bmatrix} \tag{11}$$

Calculate Eigenvalues and Eigenvectors

The relative importance (weight) of sub-criteria with respect to the criterion one level above is determined from the eigenvector of the matrix. The principal eigenvalue and the corresponding normalized right eigenvector of the comparison matrix give the relative importance of the various criteria being compared. The elements of the normalized eigenvector are termed weights with respect to the criteria or sub-criteria and ratings with respect to the alternatives.

The Eigenvector associated with the principal Eigen value of a Matrix 'A' can be calculated as (Saaty 1980):

$$\lim_{k \to \infty} \frac{A^k e}{e^T A^K e} = Cw \tag{12}$$

where e is the column unity vector and e^T its transpose, C is a constant, and w is the Eigen vector.

Check Consistency of Matrices

Comparisons made by this method are subjective and the AHP tolerates inconsistency through the amount of redundancy in the approach. If this consistency index fails to reach a required level, then answers to comparisons may be re-examined. The consistency index, *CI*, is calculated as:

$$CI = \frac{\lambda_{\max} - n}{n - 1} \tag{13}$$

where n is the order of the matrix and $\lambda_{\max}$ is the maximum eigenvalue of the judgement matrix. For a perfectly consistent matrix of pairwise comparisons, the *CI* would be zero, because the eigenvalue is equal to the order of the matrix (Saaty and Vargus 1982). The consistency of judgments in the pairwise comparisons can be calculated by finding the consistency ratio. The consistency ratio, *CR*, can be defined as the ratio of the consistency index and the random index (Saaty 1982).

$$CR = \frac{\text{Consistency Index}}{\text{Random Index}} \tag{14}$$

The Random Index (RI), for the different order random matrices was calculated by Saaty (1982) which is listed in table 6.

Table 6. Random index

Size of Matrix	Random Index	Size of Matrix	Random Index
1	0	9	1.45
2	0	10	1.49
3	0.58	11	1.51
4	0.9	12	1.48
5	1.12	13	1.56
6	1.24	14	1.57
7	1.32	15	1.59
8	1.41		

When making judgments concerning multiple comparisons, the objective behind good decision-making is not to minimize the consistency ratio. Good decisions are most often based on consistent judgments, but the reverse is not necessarily true. AHP allows a margin of inconsistency, and a *CR* of 0.10 or less, is generally considered to be an acceptable amount for inconsistency for evaluating the decision hierarchy (Saaty and Vargus 1982). If the *CR* is above 0.10, then the values assigned to the pairwise comparison are likely too inconsistent, and the results may not be meaningful.

Evaluate and Compare Alternatives for Criteria and Decision making

Determination of the best alternative is performed by evaluating the aforementioned parameters for a foundation reuse project. The final weights for sub-criteria can be obtained by multiplying the weight (from the eigenvector) with the weight of the corresponding criteria one level higher in the hierarchy. Each alternative is then evaluated for its effectiveness in achieving the objective stated in each sub-criterion using pairwise comparison, like step 2. The consistency of these pairwise matrices is checked, and eigenvectors are calculated. These eigenvectors represent the how each criterion performs. Finally, a matrix of eigenvalues obtained from the previous stage is created and then multiplied with the transpose of the final sub-criterion weights.

$$\begin{array}{r}\text{Alternativ e x}\\ \text{Alternativ e y}\\ \text{Alternativ e z}\end{array}\begin{bmatrix} e_{1x} & e_{2x} & e_{3x} & e_{4x} & e_{5x} \\ e_{1y} & e_{2y} & e_{3y} & e_{4y} & e_{5y} \\ e_{1z} & e_{2z} & e_{3z} & e_{4z} & e_{5z} \end{bmatrix}\begin{bmatrix} w_1 \\ w_2 \\ w_3 \\ w_4 \\ w_5 \end{bmatrix} \tag{15}$$

where w_1, through w_5 are the final weights for their respective sub-criteria. e_{1x} is the effectiveness of alternative x for sub-criteria 1.

Perform a Sensitivity Analysis of Model

Sensitivity analysis is performed to see how well the alternatives perform with respect to each of the criteria as well as how sensitive the alternatives are to changes in the importance of the criteria. Sensitivity analysis of the AHP-based reuse decision-making model requires comparing the change in output with changes to the input. This entails changing the pairwise comparison values for every factor and conducting the entire analysis again. This is performed for every factor at many different levels. Many commercially available software programs such as expertchoice® (can be utilized to conduct sensitivity analysis.

Hypothetical Example Using AHP-based Method

To show the application of the AHP-based method for foundation reuse decision-making, a hypothetical example is presented. The weights associated to each criterion are just to demonstrate the application of the approach. These numbers would be defined for each specific project according to the priorities and specific considerations of that project.

Step 1- Structure decision hierarchy

The hierarchy decision structure is defined as per figure 27.

Step 2- Construct comparison matrices

The main matrix consists of the highest level of criteria to fulfill the main objective. Based on the relative importance of the factors, the numerical weights are assigned according to table 5. The existing comparison matrices are shown in equations 16 to 20.

$$\begin{bmatrix} w_S/w_S & w_S/w_C & w_S/w_E & w_S/w_F \\ w_C/w_S & w_C/w_C & w_C/w_E & w_C/w_F \\ w_E/w_S & w_E/w_C & w_E/w_E & w_E/w_F \\ w_F/w_S & w_F/w_C & w_F/w_E & w_F/w_F \end{bmatrix} = \begin{bmatrix} 1 & 2/1 & 3/1 & 3/1 \\ 1/2 & 1 & 2/1 & 2/1 \\ 1/3 & 1/2 & 1 & 2/1 \\ 1/3 & 1/2 & 1/2 & 1 \end{bmatrix} \tag{16}$$

$$\begin{bmatrix} w_C/w_C & w_C/w_R \\ w_R/w_C & w_R/w_R \end{bmatrix} = \begin{bmatrix} 1/1 & 2/1 \\ 1/2 & 1/1 \end{bmatrix} \tag{17}$$

$$\begin{bmatrix} w_L/w_L & w_L/w_E & w_L/w_W \\ w_E/w_L & w_E/w_E & w_E/w_W \\ w_W/w_L & w_W/w_E & w_W/w_W \end{bmatrix} = \begin{bmatrix} 1/1 & 2/1 & 3/1 \\ 1/2 & 1/1 & 2/1 \\ 1/3 & 1/2 & 1/1 \end{bmatrix} \tag{18}$$

$$\begin{bmatrix} w_A/w_A & w_A/w_U \\ w_U/w_A & w_U/w_U \end{bmatrix} = \begin{bmatrix} 1/1 & 2/1 \\ 1/2 & 1/1 \end{bmatrix} \tag{19}$$

$$\begin{bmatrix} w_S/w_S & w_S/w_R \\ w_R/w_S & w_R/w_R \end{bmatrix} = \begin{bmatrix} 1 & 2/1 \\ 1/2 & 1 \end{bmatrix} \tag{20}$$

Step 3 – Calculation eigenvalues and eigenvectors

The eigenvalues for all the matrices are then calculated. The five-step "Average of normalized columns" method is followed to calculate the eigenvalues. Table 7 summarizes the eigenvalues that were obtained from the calculations.

Table 7. Eigenvalues calculated using average of normalized columns

Criterion	Sub-Criteria	Eigenvalue
Safety	Capacity	0.67
	Resilience	0.33
Environmental	Land use	0.54
	Energy Consumption	0.30
	Solid Waste	0.16
Cost	Agency costs	0.67
	User costs	0.33
Functionality	Service life	0.67
	Rideability	0.33

Step 4 – Check consistency of matrices

As it is shown in table 8, the consistency of matrices is checked with the consistency index, CI, and the ratio of the CI and the random index (RI) gives the corresponding consistency ratio (RC).

Table 8. Consistency values for the hypothetical example

Matrix	Consistency Index (CI)	Random Index (RI)	Consistency Ratio (CR = CI/RI)
Main Matrix	0.027	0.9	0.031
Safety Matrix	0.00	0.58	0.00
Environmental Matrix	0.004	0.58	0.008
Cost Matrix	0.00	0.58	0.00
Functionality Matrix	0.00	0.58	0.00

All matrices in table 8 have CRs less than 0.10 and are therefore consistent.

Step 5- Evaluate and compare alternatives for criteria and decision making

In this step, the pairwise comparison matrices of defined options with respect to each sub-criterion are created. For example, Eq. (21) and 22 show the pairwise comparison matrix for the capacity sub-criterion (Level 3) under the safety criteria (Level 2), and the energy consumption sub-criterion (Level 3) under environmental impact criteria (Level 2), respectively. Then, the eigenvalues obtained from these matrices are calculated and the final weights in table 7 are substituted into equation 10.

$$
\begin{array}{c} \\ Opt1 \\ Opt2 \\ Opt3 \\ Opt4 \end{array}
\begin{array}{c}
\begin{array}{cccc} Opt1 & Opt2 & Opt3 & Opt4 \end{array} \\
\begin{bmatrix} C_1/C_1 & C_2/C_1 & C_3/C_1 & C_4/C_1 \\ C_1/C_2 & C_2/C_2 & C_3/C_2 & C_4/C_2 \\ C_1/C_3 & C_2/C_3 & C_3/C_3 & C_4/C_3 \\ C_1/C_4 & C_2/C_4 & C_3/C_4 & C_4/C_4 \end{bmatrix}
\end{array}
= \begin{bmatrix} 1 & 1 & 1 & 0.5 \\ 1 & 1 & 1 & 0.5 \\ 1 & 1 & 1 & 0.5 \\ 2 & 2 & 2 & 1 \end{bmatrix} \tag{21}
$$

$$
\begin{array}{c} \\ Opt1 \\ Opt2 \\ Opt3 \\ Opt4 \end{array}
\begin{array}{c}
\begin{array}{cccc} Opt1 & Opt2 & Opt3 & Opt4 \end{array} \\
\begin{bmatrix} E_1/E_1 & E_2/E_1 & E_3/E_1 & E_4/E_1 \\ E_1/E_2 & E_2/E_2 & E_3/E_2 & E_4/E_2 \\ E_1/E_3 & E_2/E_3 & E_3/E_3 & E_4/E_3 \\ E_1/E_4 & E_2/E_4 & E_3/E_4 & E_4/E_4 \end{bmatrix}
\end{array}
= \begin{bmatrix} 1 & 1 & 3 & 3 \\ 1 & 1 & 4 & 2 \\ 0.33 & 0.25 & 1 & 2 \\ 0.33 & 0.33 & 0.5 & 1 \end{bmatrix} \tag{22}
$$

The eigenvalues of capacity and energy consumption matrices are shown in table 9 as a sample calculation. These eigenvalues are calculated for all 9 sub-criteria.

Table 9. Eigenvalues of two sub-criteria

Criterion	Eigenvalues
Capacity	0.2
	0.2
	0.2
	0.4
Energy consumption	0.375
	0.361
	0.117
	0.140

The option with the biggest weight is the preferred option in the AHP-based method.

$$
\begin{array}{c} \text{Option } 1 \\ \text{Option } 2 \\ \text{Option } 3 \\ \text{Option } 4 \end{array}
\begin{bmatrix} 0.20 & 0.36 & 0.38 & 0.38 & 0.38 & 0.40 & 0.40 & 0.11 & 0.11 \\ 0.20 & 0.36 & 0.36 & 0.36 & 0.35 & 0.34 & 0.37 & 0.12 & 0.12 \\ 0.20 & 0.15 & 0.12 & 0.12 & 0.13 & 0.12 & 0.12 & 0.37 & 0.37 \\ 0.4 & 0.15 & 0.14 & 0.14 & 0.14 & 0.13 & 0.11 & 0.40 & 0.40 \end{bmatrix}
\begin{array}{c} \textit{Weights} \\ \begin{bmatrix} 0.67 \\ 0.33 \\ 0.54 \\ 0.30 \\ 0.16 \\ 0.67 \\ 0.33 \\ 0.67 \\ 0.33 \end{bmatrix} \end{array}
\begin{array}{c} \\ \text{Capacity} \\ \text{Resilience} \\ \text{Land Use} \\ \text{Energy} \\ \text{Solid Waste} \\ \text{Agency Costs} \\ \text{User Costs} \\ \text{Service Life} \\ \text{Rideabilit y} \end{array} \tag{23}
$$

The first and fourth column of the left matrix is eigenvalues of capacity and energy consumption sub-criterion pairwise matrix listed in table 9, respectively.

$$
\begin{aligned}
\text{Option } 1 &= 0.20 \times 0.67 + 0.36 \times 0.33 + ... + 0.11 \times 0.33 = 1.14 \\
\text{Option } 2 &= 0.20 \times 0.67 + 0.36 \times 0.33 + ... + 0.12 \times 0.33 = 1.08 \\
\text{Option } 3 &= 0.20 \times 0.67 + 0.15 \times 0.33 + ... + 0.37 \times 0.33 = 0.79 \\
\text{Option } 4 &= 0.40 \times 0.67 + 0.15 \times 0.33 + ... + 0.40 \times 0.33 = 0.98
\end{aligned}
\tag{24}
$$

As can be seen from results above, Option 1 is the preferred alternative among all other alternatives in this hypothetical example.

INVEST® (FEDERAL HIGHWAY ADMINISTRATION)

The Federal Highway Administration (FHWA) has developed a web-based tool for the evaluation of the sustainability of transportation projects: Infrastructure Voluntary Evaluation Sustainability Tool (INVEST, https://www.sustainablehighways.org). The evaluation is done through a series of criteria which cover the full lifecycle of transportation services, including system planning, project planning, design, and construction, and continuing through operations and maintenance. System planning for States (SPS), system planning for regions (SPR), project development (PD), and operations and maintenance (OM) are four scoring modules for transportation projects provided by INVEST. Each of these modules contains several criteria which enable decision-makers to evaluate their plans, projects, and programs. Each INVEST criterion has a description about the goal of the criterion, linkage to the sustainability area, the scoring requirements for receiving points, sources of supporting documentation, and links to additional information resources.

INVEST could be a potential tool for decision-making in foundation reconstruction/reuse, both on a system planning level (SPS) and a project development (PD) basis. The current scoring system in the SPS module includes can be impacted directly by foundation reuse in the areas of “demonstrating sustainable outcomes” (SPS-1) and “financial sustainability” (SPS-12). Successfully completing reuse projects that produce lower costs and sustainable outcomes can improve the scoring for these modules. The PD module includes scoring for items such as historic preservation (PD-15), reduce, reuse, repurpose (PD-19), recycle (PD-20), earthwork (PD-21), construction equipment emission reduction (PD-25), noise (PD-26), and construction waste (PD-29). Although foundation reuse alternatives can present scoring advantages, inclusion of additional areas in future would allow the benefits of foundation reuse to be fully accounted for. Some such areas include reduction to required amount of earthwork, reduction in construction equipment emissions, reduction to amount of construction noise generated, and reduction of construction waste.

CHAPTER 3. PRELIMINARY DESK STUDY

INTRODUCTION

The evaluation of a foundation for potential reuse begins by accumulating as much existing data on the foundation as possible. This step, referred to as the desk study, is a cost-effective method of information collection. The desk study can reveal the existing gaps in available information and will provide a preliminary determination of the current state and past performance of the foundation. This data can include design drawings, inspection reports, test data, and maintenance records, and can be helpful in determining the tests that need to be performed to ensure that the foundation is in adequate shape, has the required capacity, meets LRFD requirements, and will perform for the intended lifespan of the replacement bridge. The primary questions for the desk study phase are:

- What are the primary details of the foundation system?
- What is the performance history of the foundation system?
- What further investigations are needed for the foundation?

Detailed investigation is carried out based on the findings of the desk study. The knowledge from these two studies is combined to assemble all the required information for the foundation reuse, as illustrated in figure 28.

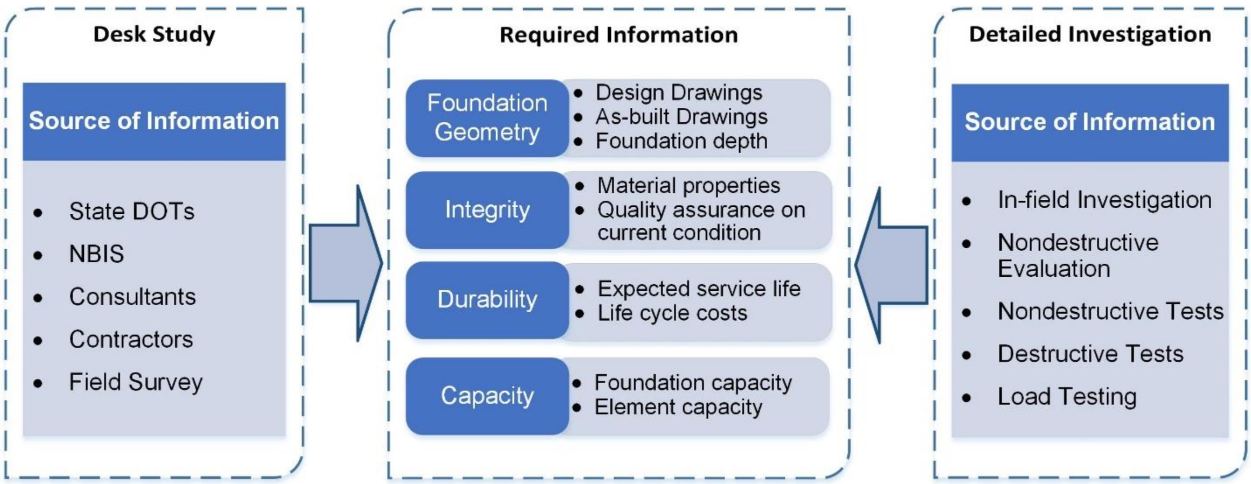


Figure 28. Illustration. Required information for a comprehensive reuse assessment

CONFIRMING PRIMARY DETAILS

As previously mentioned, the primary details of an existing bridge foundation consist of its dimensions, locations, and depth. Important documentation that can be used to identify these details consist of:

- Design plans
- Installation records
- Soil properties and profile (soil borings and testing)
- Depth of drilled shafts/ driven piles (design/ as-built)

Design Plans

A full set of original design plans and drawings is crucial to the evaluation of foundations for reuse. Important details that may be gleaned from design plans include foundation geometry, design material strengths, rebar layout and details, design pile length, and pile capacity. While important, design plans may not necessarily reflect the final configuration of the structure, as changes made during construction may not have been fully documented. Often, older substructures were designed using allowable stress design (ASD) or other older approaches. The ASD capacity found in these plans does not translate to an LRFD capacity, as the factors and sometimes even methods of determining capacity vary between the two design approaches. The review of the design plans can help determine the conditions of the structure and underlying soil.

Some details obtained from construction drawings will naturally be more consistent with the actual constructed state than others. Overall confidence in the accuracy of the details provided by the design drawings is project specific and relies on a combination of engineering experience and observations. Initial review of design documents often includes verifying easily observed details such as above ground geometry with the documentation found. If differences exist between the documentation found and easily observable details, greater care may be required to ensure that unobservable details such as underground geometry, rebar layout, and pile depth are consistent with the design plans.

Some foundations lack design drawings, plans, as-built records, and driving logs. These foundations are often referred to as "unknown foundations" (Schaefer and Jalinoos 2013) and considering them for reuse will require significantly greater expenditures for field investigation including NDT, geophysical methods, etc. What is more common than "unknown foundations" are bridges where design drawings are available, but not in full. For instance, drawings may show reinforcement size and layout, but may not show lap splices details, development length details, etc. In these circumstances, additional nondestructive evaluation will be required to ensure that the rebar layout is compliant with modern codes. Bridges with non-compliant details may require strengthening (FRP wrapping, insertion of dowels, etc.) to achieve the requisite capacity and appropriate service life.

Installation Records

Installation records provide additional confidence to what the in-situ details of the substructure are. "Installation records" is a broad term used here to describe as-built drawings, revised design plans, pile driving logs, rebar inspection logs, or other field reports from the time of installation. While these records can be particularly helpful, they are often not available when evaluating a foundation for reuse. When available, they provide increased confidence in the exact details of the structure and less investigation is required to confirm the installed geometry of the substructure.

As-built drawings generally exist either as a separate set of drawings, or as a set of revisions to the original designs. These drawings or revisions document the actual constructed state of the bridge, and how it deviated from the designed state. These drawings, in some form, are a requirement for new construction. For older bridges, these records are often missing, reducing confidence that the foundation is constructed as designed.

Pile driving logs are highly valuable, but often have missing or incomplete information. The required amount of detail for pile driving logs of new construction is generally well defined by State DOTs, but older bridge files may contain variable amounts of detail. The lowest level of details found in pile driving logs is the as-built pile depths. Knowledge of this depth is crucial in ensuring capacity and stability against scour floods, liquefaction, etc. The next important piece of information is the End-of-Driving (EOD) records for production and test piles. Having an EOD record for each pile ensures that the pile was driven to an appropriate resistance. When coupled with test data, the EOD record ensures that the criteria used to drive the test pile was repeated with the remaining production piles. Pile driving logs that include blow counts for the entire pile driving help ensure that the piles were not damaged during installation by driving through hard layers or over-driving in soft layers. Local geotechnical experience and familiarity with past practice is very beneficial when weighing the appropriate level of concern about installation damage.

Installation of a new drilled shaft requires typically involves oversight and quality control by engineers by documenting the depth of bedrock and the elevation of the bottom of excavation and verifying that the bottom of excavation was free of loose material, a tremie tube was used to pour concrete in wet or dry construction, and the rebar cage was as designed and properly installed. In many States, Crosshole Sonic Logging (CSL), as described by ASTM D6760 (ASTM 2016e), is required to be performed after installation to ensure the quality of the constructed shaft. For older drilled shafts, it is unlikely that this testing was performed and even more unlikely that the results from this testing were retained.

Rebar inspection logs may not exist for older existing foundations, as this would require extensive installation documentation. When inspection records of the rebar installation are available, engineers can be reasonably confident about the capacity of reinforced concrete sections. Without these records, engineers will may need to perform inspections to ensure that the design reinforcement is in place, and that splices are sufficient. The design cover depth will generally be available along with the provided documentation, but the depth of concrete cover tends to be a highly variable number, even within a single element. When durability risk factors, such as chloride intrusion or carbonation are present, measurement of the in-situ cover depth can help provide more a robust analysis of the element’s durability.

Soil Exploration

Along with the original design plans, soil boring logs and testing conducted for the original project can often be found. The most common information that can be found in these records is boring logs (soil type, gradation, soil profiles), and SPT data. Foundations supported by rock may contain records of rock coring logs and testing. These records can aid in the understanding of the type of soil and rock underlying the foundation, regardless of whether the foundation is shallow, pile-supported, or on drilled shafts. The design foundation capacity (bearing capacity for shallow foundations, total capacity for deep foundations) can often be found or estimated from original design documents. The soil profile is typically confirmed with at least some additional soil exploration during the integrity investigation.

Installed Pile/Drilled Shaft Length

Driven Piles

When pile driving logs from the original pile installation exist, the engineer investigating reuse feasibility can be reasonably certain about the depth piles were driven to. If pile test data exists without driving logs, an installed length may be available on the test logs, but only for the tested piles, which may be a small fraction of all the piles. Design plans will generally have the design pile lengths, but actual driven lengths will typically vary from the design value due to uncertainty in field conditions. Determining the in-situ pile depth is good practice for piles in all soil conditions, although the criticality of knowing the in-situ depth is dependent on the subsurface details. For piles driven to bedrock, an important consideration is confirming that piles were socketed to bedrock. For foundations in scour-prone areas, knowing the depth of the foundation is extremely important when evaluating pile capacity and stability under potential future scour.

Drilled Shafts

Drilled shafts are often accompanied by detailed installation logs documenting the depth of installation, the drilling conditions, and various quality control measures. Without these records, verification of the length of the drilled shaft may be necessary. Techniques like Sonic-Echo (SE) or Impulse Response (IR) can be used to estimate shaft length without direct measurements. However, coring through drilled shafts to the underlying soil or rock with a core barrel attached to a standard drill rig allows for confirmation of the depth of the drilled shaft, identification of defects, material properties of concrete, and observation and testing of the soil/rock conditions underlying the bottom of the drilled shaft. Information obtained through this approach will be more reliable than SE/IR methods as it provides detailed information on the current condition of the shaft.

ASSESSING PAST FOUNDATION PERFORMANCE

Inspection Records

Inspection records provide documentation of the past performance and condition history of the entire bridge system, including superstructure and substructure. The AASHTO Manual for Condition Evaluation of Bridges (AASHTO 2016) lists seven different types of inspections that can be performed on bridges:

- Initial Inspections
- Routine Inspections
- Damage Inspections
- In-Depth Inspections
- Fracture-Critical Inspections
- Underwater Inspections
- Special Inspections

Initial and routine inspections will provide a time history of the deterioration of visible elements (pier caps, walls, exposed piles, pier columns, etc.). Damage inspections are performed after extreme events such as impact, collision, fire, flooding, seismic, etc. Even damage that was repaired may have resulted in changes to the structural behavior. The damage inspection report is crucial in determining if this is the case. In-depth inspections may have been performed in the past at arm's length to ascertain the state of an element. In fact, any nondestructive testing performed to ascertain the integrity, capacity, or durability of the foundation for reuse can be considered an in-depth inspection. Fracture critical inspections are performed on elements whose failure could cause collapse of the bridge. These are members with non-redundant load paths, which can be part of the substructure when steel transfer girders are used to connect girders to pier columns outside of the superstructure. Underwater inspections of piles or bridge piers will help in understanding the condition of underwater elements and their exposure to hydraulic scour. Special inspections refer to inspections used to monitor scour, settlement, fracture critical cracks, or to monitor the condition of a specific section or element.

Routine Inspection Data

Records from routine bridge inspections help assess how the structure has performed and evolved over time. Inspection reports typically include condition numbers for members of the exposed substructure, as well as details on the size, extent, and seriousness of the deficiencies noted during these inspections. When the review of this documentation is performed during the earliest steps of the reuse investigation, the information obtained can help guide where additional exploration, evaluation, and testing is required.

The routine inspection history of concrete members documents the presence of and extent of cracking, scaling, and spalling over time. Signs of efflorescence and rust staining may have been noted during the inspections. Documented rust staining can be used as an indication of the timing of corrosion initiation, though corrosion can occur without rust staining. Deterioration that began or rapidly accelerated recently will have had less of an impact on the reinforcement steel than deterioration and corrosion that has long existed.

Routine inspection of steel substructure members may have identified and monitored any corrosion present. Examining inspection reports from regular intervals can help show how deterioration has progressed over time. The performance and integrity of any coating or protection system, including paint, coal tar epoxy, galvanization, or cathodic protection can be understood from these inspection reports. As with concrete elements, establishing the time history of this deterioration is important and can only be achieved by studying the long-term inspection data.

Inspection data on timber elements typically documents exterior issues that could have led to the onset of deterioration. Major issues to be considered include the presence of cracks or splitting, as these would allow funguses and insects to reach beyond the exterior preservatives to the center of the pile. Other important issues include excessive deflection of elements or loose connections. If as-built plans are not available, visual inspection can confirm that the substructure was built as designed, including issues such as pile batter (or lack thereof), pile diameter, and presence of knots or other features.

The inspection history of masonry elements typically documents the condition of the masonry blocks and the condition of the mortar being used to bind the blocks together. Reuse of foundations which incorporate rubble masonry (masonry with binding mortar), is uncommon and may warrant additional evaluation. Since mortar will generally have a much lower compressive strength, degradation of the available capacity in the joints can lead to unsafe conditions. Water seepage through a masonry wall can be an indication of issues with cracking or mortar issues. If seepage is continually eroding mortar with it, a white residue may be visible on the outside of the masonry.

Underground elements are generally not inspected during routine inspection. For these elements (driven piles, drilled shafts, pile caps, footings) excavation would need to be performed for visual inspection. Often, underground elements are subject to less harsh conditions than their above ground counterparts. In these cases, the expected condition of the underground elements is generally inferred from the observed condition of above ground elements during routine inspections. However, relying on only above ground observations may not appropriately address all concerns related to the safety and durability of a reused foundation.

Damage Inspections

Documentation is often available for members that have become damaged during the lifespan of the bridge. Damage to substructure elements can include impacts, fire damage, earthquake damage, or scour damage. The extent of this damage can be found in inspection logs from damage inspections at the time of occurrence. Documentation of the design calculations behind any repairs performed would allow for the capacity of the repair to be reevaluated prior to reuse.

Underwater Inspections

Underwater elements may have been periodically inspected to determine their condition and integrity. These inspections are not generally performed at the same regular interval as routine inspections. The underwater inspection will document if deterioration is occurring at a different rate below the waterline than above it. The scour depths of the existing elements will generally be documented during these inspections. Underwater inspection will most likely have been performed on foundations where inspection was relatively simple, generally in cases where the underwater components could be waded out to and inspected without the need for diving equipment and support. Inspections on foundations which require divers are likely to have only been conducted when there is suspicion that damage related to floods, scour, or other hazards has occurred.

ASSESSMENT PROCEDURE

This research divides the major concerns to be addressed during evaluation into three categories: integrity, capacity, and durability (see table 10). Each of these three categories will follow from a complete review of the records available for the bridge, including design plans, installation records, soil exploration history, and inspection records. These assessments begin with fact-finding investigation of the existing structure, progress to identify areas where knowledge is lacking, then taking the appropriate steps toward eliminating gaps in knowledge.

Table 10. Assessment process for a foundation reuse candidate

Assessment Portion	Tasks
Desk Study	Collect and review design drawings, installation records, soil boring history, soil test data, QA/QC records, inspection history, hazard history, and other reports.
Integrity Assessment	Determine material properties of foundation structures. Assess component damage and deterioration. Identify uncertain details, such as pile length and subsurface dimensions. Evaluate geotechnical performance, including settlement, geo-hazards, slope stability, and other changes to geotechnical system.
Durability Assessment	Assess current state of the bridge and level of deterioration. Assess environmental factors at the bridge that may lead to future deterioration of elements. Estimate remaining service life. Identify potential life cycle costs of durability issues identified and the life cycle costs of repair measures identified.
Capacity Assessment	Determine new loads on the foundation. Determine capacity of existing components, accounting for integrity and durability assessments. Determine capacity of footings and deep foundations, performing load testing if necessary.

The integrity assessment primarily focuses on identifying any damage, deterioration, or adverse conditions that may have impacted the intactness or capacity of the current structure. Integrity assessment begins with understanding the structural and geotechnical systems in place, how they function, and what potential areas of concern could arise. The integrity assessment includes a review of all documentation related to the substructure, including inspection reports, previous repairs, and any extreme loading conditions reported, including flooding and scour, geo-hazards, seismic, fire, and impacts. After potential deficiencies or modes of failure have been identified, additional testing may need to be performed to help accurately determine the in-situ conditions.

After the current condition of the substructure is ascertained during the integrity assessment, the assessment into the future lifespan of the substructure can be conducted. This assessment is hereafter referred to as the durability assessment and includes assessing the remaining service life and the future life cycle costs of the substructure. Durability issues can include corrosion of reinforcing steel or steel elements, spalling, abrasion, rotting of timber elements, degradation of masonry joints, and other forms of progressive deterioration. Inspection logs and local experience are invaluable in determining the durability issues that need to be considered prior to reuse. Bridge inspection reports taken at regular intervals are important as they provide a time history of the condition and performance of the structure. Various nondestructive test (NDT) procedures are available to help determine the current conditions of the substructure-, a requirement of anticipating future conditions.

Capacity assessment builds on the information gathered in the integrity and durability assessments to determine the reliable capacity available. As with the integrity assessment, the initial steps include a thorough review of all data available. Pile test logs, driving records, concrete test breaks, rebar layout plans, and all other structural details provide meaningful insight into the capacity of the foundation. NDT/NDE can be used to fill in gaps from missing, incomplete, or unreliable data, to establish with certainty what structural conditions and material properties are present. Important quantities such as pile capacity, shallow foundation bearing capacity, and serviceability (deflection criteria) of the foundation are included in the capacity assessment.

While there is some hierarchy to the assessments, in many cases the testing required for each assessment can be performed simultaneously, or out of order. If one assessment phase is expected to control the suitability of the foundation for reuse, it may be prudent to investigate that aspect prior to performing other aspects. In many cases, foundations will be suitable for reuse, but will require repair, strengthening, or retrofitting. Repaired or strengthened sections may have their own durability considerations that impact the overall durability of the foundation. Typically, several reuse alternatives can be developed early in the reuse process that can be compared in terms of suitability and feasibility. A flowchart of the assessment procedure for reuse of foundations is shown in figure 29.

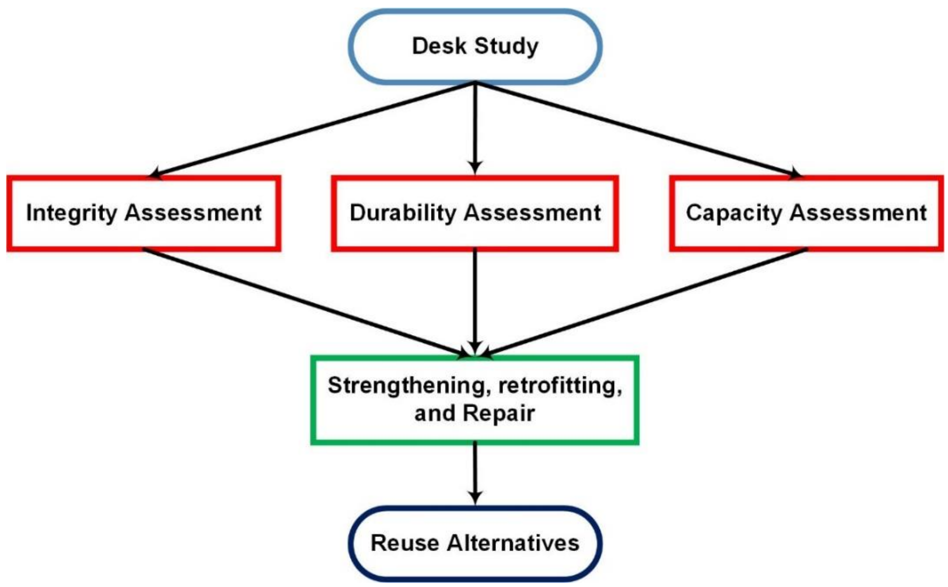


Figure 29. Chart. Reuse assessment procedure

CHAPTER 4. INTEGRITY ASSESSMENT

INTRODUCTION

This chapter discusses the assessment of structural and geotechnical integrity of existing foundations being considered for reuse. This assessment involves a combination of visual observations, sample testing, non-destructive testing (NDT), and vertical coring with wireline logging. Several of the case examples mentioned in Chapter 1 that involved substantial integrity investigation are discussed in further detail.

The most obvious difference between a new bridge foundation and a reused one is the age-related deterioration and loss in load capacity. Load and Resistance Factor Design (LRFD) is for the most part analysis at the construction level, meaning that the design calculations are performed using the properties of the structure on the day it was placed into service. Existing substructure elements being considered for reuse have been exposed to the environmental elements, potentially lack critical documentations, and were not necessarily constructed with QA/QC practices consistent with modern code requirements. To produce an existing foundation re-design compliant with modern codes, confidence in the initial build quality and the current condition of the substructure is crucial. The major questions addressed during the integrity evaluation are:

- What are the material properties of the existing elements?
- Has deterioration reduced the current capacity of the foundation?
- Has the foundation been damaged (geo-hydraulic hazards, impacts, etc.)?
- Are there changes to soil system or stability issues?

Answering these questions involves evaluation of structural components above and below the ground surface. These elements may be constructed from a variety of materials, including concrete, steel timber, and masonry. In addition, the integrity assessment involves evaluation of relevant geotechnical issues, including past performance, geo-hydraulic hazards, soil properties, and liquefaction potential. The flowchart in figure 30 shows aspects of integrity assessment that may be considered during a reuse investigation for different material types.

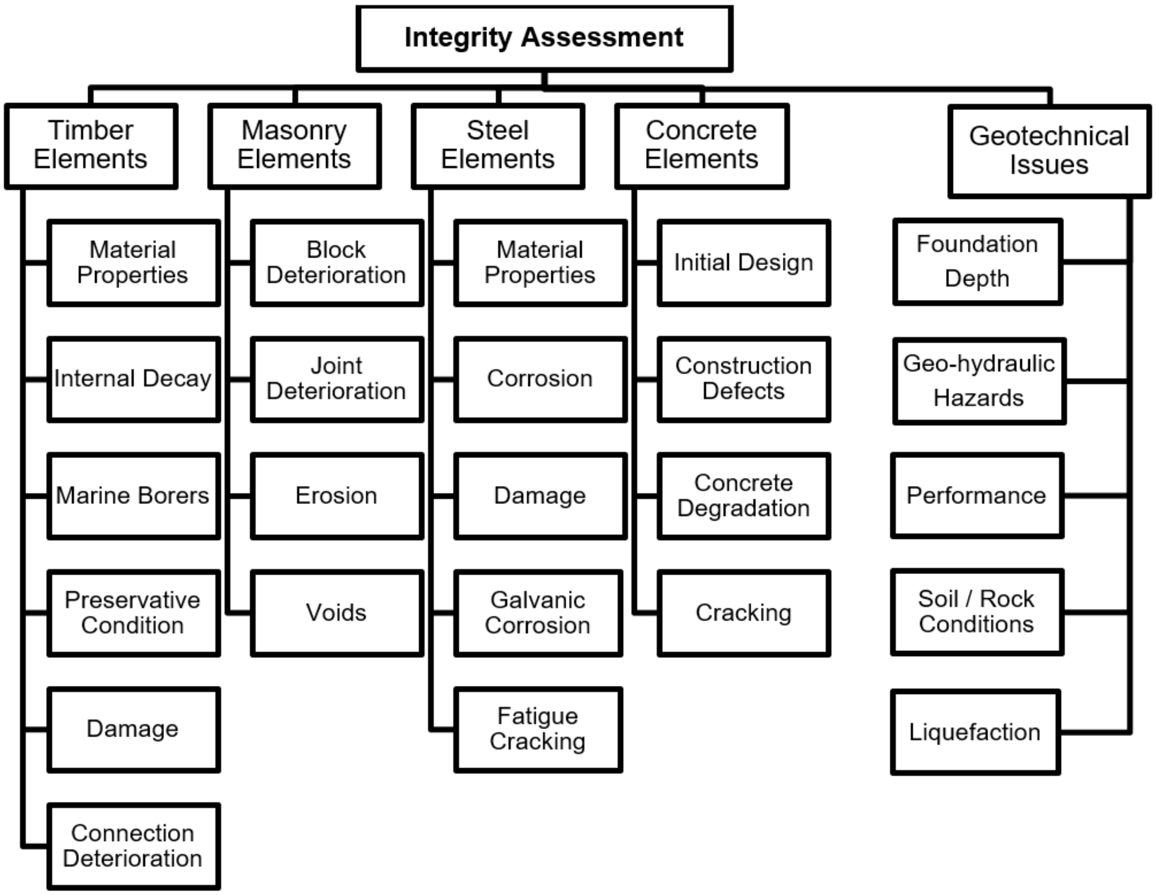


Figure 30. Illustration. Components of integrity assessment

STRUCTURAL INTEGRITY OF ABOVE GROUND ELEMENTS

Concrete Element Issues

Concrete is one of the most common building materials in bridge substructures. Reinforced concrete is the primary building material used for pier walls, columns, pier caps, footings, abutment walls, piles, and drilled shafts. Precast prestressed concrete sections are commonly used as driven piles. While concrete possesses high compressive strength, it has very little tensile strength and typical has rebar, prestressing strands, or post-tensioning bars as tensile reinforcement. The encasement of the steel reinforcement in concrete provides protection for the steel from corrosive elements; although this protection can be lost if the steel becomes exposed, or the concrete becomes excessively cracked or deteriorated. The corrosion of this steel is typically the controlling factor when determining the service life and durability of reinforced concrete elements. As such, the primary discussion of steel corrosion is provided in Chapter 5, and the integrity assessment focuses on evaluating the initial design, construction defects, degradation, damage, and cracking of concrete elements.

Initial Design

Important aspects of the initial concrete design include mix properties, compressive strength, rebar layout, rebar strength, and geometry of underground components.

Mix Properties

New bridge construction will typically use concrete mixtures designs preapproved by State and local agencies. These designs will often have a history of suitable performance and will be designed to withstand the expected conditions over the service life of the bridge. For an existing bridge, various aspects of the concrete mixture may be unknown: the water/cement ratio, aggregate details, the amount of entrained air, the size and dispersion of entrained air, and the permeability of the concrete. Beyond knowing the compressive strength, understanding these properties is important for accurate prediction of freeze-thaw susceptibility, strength, and design life predictions. Integrity assessment can either confirm findings from mixture design documents or fill knowledge gaps by determining these properties in-situ.

Compressive Strength

The strength of concrete is generally defined by its unconfined compressive strength. New concrete elements will be subjected to testing to ensure adequate strength, but this data may not be available for existing foundations. Table 6A.5.2.1-1 of the MBE (AASHTO 2016) provides minimum compressive strengths of concrete by year of construction. While these minimums are acceptable for load rating, a more thorough investigation can provide higher confidence in the in-situ conditions, allowing for less conservative strength values than those in the MBE (AASHTO 2016). ACI 437R (ACI 2003) recommends combining core sampling/testing with “in-place tests,” such as penetration testing, rebound testing, or acoustic wave testing when evaluating the strength of existing concrete members. In addition, coreholes and wireline logging allow for tests of the concrete strength (on extracted cores) that can be correlated to continuous density or wave speed logging. Bungey et al. (2006) provide a comparison of the relative merits of strength tests, as shown in table 11.

Table 11. Relative merits of strength tests (Bungey et al. 2006)

Test Method	Cost	Speed of Test	Damage	Representativeness	Reliability of absolute strength correlations
Cores	High	Slow	Moderate	Moderate	Good
Pull-out	Moderate	Fast	Minor	Near Surface Only	Moderate
Penetration resistance	Moderate	Fast	Minor	Near Surface Only	Moderate
Pull-off	Moderate	Moderate	Minor	Near Surface Only	Moderate
Break-off	Moderate	Moderate	Minor	Near Surface Only	Moderate
Internal fracture	Low	Fast	Minor	Near Surface Only	Moderate
Ultrasonic pulse velocity	Low	Fast	None	Good	Poor
Surface hardness	Very low	Fast	Unlikely	Surface Only	Poor

Core removal and testing, as discussed in the “Sample Testing” section, is the only test method that directly measures compressive strength. The remaining methods are nondestructive or partially destructive and do not provide as accurate estimation of absolute concrete strength. Instead, these methods are often used in conjunction with compressive strength testing to identify weaker areas of concrete and estimate the strength relative to the tested cores. Vertical coreholes performed with accompanying wireline logging allow for direct samples of concrete to be made at discrete points with an accompanying continuous vertical log of density and wave speed. These methods allow for more extensive and rapid sampling of relative concrete strength, allowing for identification of potential weak zones.

Reinforcement Layout

Rebar depth is a common concern for durability purposes, and other details such as bar size, number of bars, lap splice length, transverse reinforcement, and confinement in connection regions are crucial to the capacity of reinforced concrete sections. In general, the amount of investigation into the rebar layout is dependent on the quality of information available. If as-built plans and inspection logs are available, little to no investigation of the rebar details may be necessary. If only design plans are available, some investigation may be warranted to confirm that the design details match the in-situ conditions.

Various technologies, such as Ground Penetrating Radar (GPR), radiography, and covermeters are commonly employed to locate reinforcement in concrete. Radiography uses a radiation emitting isotope and film to determine the location of rebar and cracks within the concrete. Covermeters function by using Magnetic Field Disturbance (MFD) and are primarily used to assess the depth to reinforcement. Advanced covermeters are capable of estimating bar size and location, although results may be variable.

Reinforcement strength

The yield strength of the rebar is usually found from the design drawings. Table 6A.5.2.2-1 of the MBE (AASHTO 2016) provides yield strengths for rebar by the year of construction. The yield strength of steel bars does not typically exhibit as much variability as concrete compressive strength, so this minimum value may often be sufficient for design. The in-situ yield strength can be determined through sample removal and testing if it is not known from the design drawings.

Issues with Initial Construction

Honeycombing

Honeycombing refers to areas of concrete where aggregate is not properly mixed with the cement paste. These areas were formed during construction due to insufficient or poorly sized aggregate. These zones typically have lower strength than surrounding properly mixed areas due to the reduced amount of paste bonding aggregate together.

Poor Quality Concrete (Weak Zones)

Like honeycombing, weak zones are typically formed in areas of improper mixing. These areas can have a higher w/c ratio than the surrounding concrete, leading to a lower compressive strength.

Cold joints

Cold joints (lift lines) form when placement was performed after previous concrete had hardened and are visually identified as lateral bands separating distinct concrete layers. Cold joints are most common in pier stems and other large elements where concrete was poured in several sections with formwork movement in between pours. Cold joints can become focal point for paste erosion, can have locally weak zones due to mixing difficulties, and create a discontinuity plane where shear failures can occur.

Voids

Voids occur due to improper mixing and concrete mixtures that are incapable of flowing around reinforcement. Voids may have existed in concrete elements for many years without being noticed, while still reducing the nominal capacity of the element.

Concrete Degradation and Damage

Concrete degradation refers to long-term processes lowering the concrete quality, like freeze-thaw damage, Alkali-Silica Reactivity (ASR), Delayed Ettringite Formation (DEF), external sulfate attack, paste erosion, and calcium leaching. Chloride ingress and carbonation can be serious issues that primarily impact the durability, as discussed in Chapter 5. Various forms of concrete degradation, as listed below, are mostly prevalent in the above the ground-level portion of the foundation/substructure (such as pile bents, wall foundations, or pneumatic caissons); although all forms of degradation can also occur in buried portions.

Freeze-thaw damage

Freeze-thaw damage is the result of water getting trapped in surficial layers of concrete, then freezing and expanding. The primary defense against freeze-thaw damage is proper mixture design of the initial concrete. Important quantities to consider are the amount of entrained air, the structure of the air void system, and the water/cement ratio. These quantities are controlled through appropriate mixture design during placement, and petrographic analysis can determine the in-situ properties.

ASR

A small amount of ASR can be found in many concrete mixtures, although often this reactivity does not substantially impact the health of the structure. When a substantial amount of ASR is occurring in a concrete mixture, the associated cracking can lead to long-term durability issues or even element integrity issues if the reactivity is severe enough. *In general, concrete elements with significant ASR are not good candidates for reuse.* The best way to diagnose ASR is by visually identifying areas of gel formation through petrographic analysis. Cracking around gel formations indicates that the ASR is significant enough to actively cause damage to the structure of the concrete.

DEF

Delayed Ettringite Formation (DEF) occurs over the life span of the structure but is due to poor construction techniques. Normally, ettringite formation occurs in the initial hours after concrete is poured and is an integral part of healthy concrete curing. When concrete curing occurs at high temperatures, the ettringites are dissolved back into the cement paste. After the paste has hardened and the concrete has cooled, the ettringites can form again. Since the concrete paste has hardened prior to ettringite formation, it can lead to cracking of the cement paste. DEF can be identified through petrography by identifying cracking around ettringite formations.

External sulfate attack

Concrete degradation can also be caused by attack from sulfates dissolved in water that the element is submerged in. The sulfates can be naturally occurring in seawater, can come from dissolved soil particles, or can originate from biological activity in sewage discharged into rivers.

Paste erosion

One common deficiency noted in older concrete is paste erosion on the exterior faces, where the cement binding agent breaks down and dissipates. This leaves the aggregate originally bound by the paste more exposed to further erosion. Paste erosion is generally a slow process but can reduce the amount of concrete available for load-carrying and can reduce the amount of cover available.

Calcium leaching

Calcium leaching generally occurs after cracking has been initiated and water is able to infiltrate and seep out of elements. The leaching is generally observable as efflorescence (white residue) on the exterior concrete faces. This residue is an indication that a loss of cement paste is occurring inside of the element.

Fire Damage

During their initial lifespan, above ground foundation elements can be exposed to fires from burning vehicles or structures. Fire damage can lead to extensive cracking at the face of the concrete, a loss of concrete strength, and buckling of rebar. Areas of extensive micro-cracking will show much less rebound than intact concrete. Ground Penetrating Radar (GPR), covermeters, and other methods of identifying rebar position can be employed to determine if buckling of the rebar has occurred. Petrographic examination allows for the extent of cracking to be measured. Significant cracking will increase the permeability of the concrete and impact its durability. Areas that have been exposed to excessive heat will show a pinkish color, and this area of concrete are not be regarded as intact and available for resistance (PCA 1994).

Cracking

Cracking generally impacts the cover concrete and can occur from shrinkage, temperature changes, flexure, shear, ASR, DEF, foundation movement, seismic demands, or reinforcement corrosion (from chlorides, carbonation, or lack of protective cover). Above ground elements can be visually inspected for cracking, but below ground elements require excavation, NDT, or geophysical logging from coreholes to observe cracking. Repairing observed cracking prior to foundation reuse is generally good practice, as reinforcement corrosion is significantly more likely when cracks are present.

Early age cracking (shrinkage, thermal)

Thermal and shrinkage cracking usually occur as part of the curing process. These cracks allow for ingress of water, chlorides and contaminants, but usually do not substantially impact the element capacity. This type of cracking does not generally present long-term concerns after repair.

Concrete degradation cracking (ASR, DEF, freeze-thaw)

Cracking related to concrete degradation also does not usually substantially impact the capacity of concrete elements. Still, this form of cracking is an indication of fundamental issues with the concrete mixture that can enable water and chlorides to reach the reinforcement level. If the cracking cannot be repaired or further cracking is expected, reuse may not be a viable alternative.

Structural cracking (flexural, shear)

Structural cracking from flexure or shear forms in tensile areas of concrete elements. Since the tensile capacity of reinforced concrete is derived from the reinforcement, this cracking does not affect the capacity of the element. Significant cracking, however, is an indication that the element has been overloaded in the past. Structural cracking is typically repaired during foundation reuse, as these cracks can present a long-term durability concern.

Foundation movement

Foundation settlement and translation can cause tensile stresses to develop in concrete elements, leading to cracking. Other than the loads imparted on the element, this cracking is usually not a structural concern because the cracks occur where the concrete is in tension. However, these cracks can allow for water and chloride ingress, impacting the durability of the element. If differential movement continues to occur after reconstruction of the bridge, the lifespan of the new bridge can be impacted. Determining the underlying source of movement allows for an informed decision on the risks it poses to the foundation after reuse.

Seismic and extreme event cracking

Seismic events can cause ground movements and overloads that lead to concrete cracking. This cracking can impact multiple faces of the element, as seismic loading tends to cause excessive tension and compression in many different areas. This cracking can lead to substantial damage of the cover concrete, and potentially even damage core concrete.

Corrosion related cracking

Rebar corrosion will eventually lead to cracking, delamination, and spalling of the cover concrete due to the expansive nature of corrosion. The underlying concern with this type of cracking is the ongoing corrosion of the reinforcement. If the corrosion cannot be halted, foundation reuse may not be feasible.

Steel Element Issues

Material Properties

Ideally, design drawings and plans will include the specification and grade of steel elements used during original construction. If there are still uncertainties about important properties like the yield strength or the ultimate tensile strength, these values can be obtained through testing. Table 6A.6.2.1-1 of the MBE (2016) provides minimum mechanical properties of structural steel based on the year of construction. These values may be suitable for load rating or preliminary analysis, but redesign for reuse may require testing to confirm material strengths.

Corrosion and Section Loss

Corrosion impacts the integrity, durability, and capacity assessments. During the integrity assessment, it is important to establish where corrosion is present and how much section loss has occurred. Corrosion is most likely to occur at the water level and alternatingly wet areas such as tidal zones, changing river heights, and changing lake levels and can often be identified from above water observations, if inspections are timed for low-tide or low water level. Steel elements that are underwater or in alternatingly wet/dry environments can also be subject to microbe-induced corrosion (Browne et al. 2010a). Microbial corrosion can be differentiated from normal corrosion by an orange patina that forms outside of the corrosion products.

Galvanic corrosion can occur in underwater elements, when the cathodic potential of the soil and concrete mass forms a corrosion cell with the steel, leading to corrosion of the steel. Often the most severe galvanic corrosion occurs just below the concrete cap (Browne et al. 2010a). When two different metals are connected to each other underwater, they can form a corrosion cell that can greatly increase the rate of corrosion on whichever metal acts as the anode.

Overloads/Damage

Steel foundations that have been subjected to impacts, seismic events, or other overloads may have undergone yielding and permanent deformation in their original service life. These deformations will cause permanent residual stresses in the section. Except when the damage is minor, steel elements with permanent residual stresses due to yielding are not candidates for foundation reuse.

Fatigue Cracking

Fatigue cracking may exist on connections and details that experience tensile or shear loading during their lifespan. Since foundations are primarily in compression, these details do not often control the capacity or service life of steel piles. Still, it is important to identify any areas which may be prone to this failure mode.

Timber Element Issues

The initial integrity assessment for timber elements assesses whether fungal growth or borer attack has occurred and determines the condition of preservatives. The most at-risk portions of timber structures will be the portions close to the water line, in an intertidal or intermittently wet zone, or right above or below ground surface.

Material Properties

Testing is not frequently performed on timber elements to determine material properties. Instead, typical values are commonly used based on published values or local experience. The highly variable nature of the strength and elastic modulus of timber elements typically precludes relying on test data to establish the strength of specimens.

Typical values of timber strength can be found in the Timber Pile Design and Construction Manual (Collins 2015), ASTM 2899 (2012c), ASTM 2555 (2016d), and the MBE (AASHTO 2016). It is good practice to consider multiple sources of information to ensure that the selected values of strength conform well to all available guidance. Local experience that pertains to the strength of locally used timber piles is also crucial.

Internal Decay

The most common form of timber deterioration is internal decay due to fungal attack. The fungi typically impact interior portions of the timber elements first, in part because preservatives are applied to pile exteriors. Decayed areas of timber experience a drastic loss in stiffness, and so are not available to carry stresses during loading.

Marine Borers

Marine borers are insects that bore through and consume the timber sections, reducing the overall cross-section. These attacks most commonly occur right at the water line. Local experience is highly valuable at this stage, as it can ascertain at an early stage what types of borers may be present, what preservatives are effective against them, and what tests will be the most useful in confirming their presence or lack thereof.

Preservative Condition

Preservatives such as creosote, chromated copper arsenate (CCA), or ammoniacal copper zinc arsenate (AZCA) are commonly used to prevent decay of timber elements. The specific choice of preservative is dependent on the type of wood used for construction, the location of the bridge, and the era during which the piles were installed. Some preservatives, such as Creosote, are not commonly used for new construction, but will be found in older construction. The type of preservative used can be obtained from documentation or field investigation. Coring of the timber elements is an easy way of verifying that preservative has appropriately seeped into the outer layers of the timber. Timber elements that have poor preservative application or cracks, holes, and other discontinuities are more susceptible to biological attack.

Damage

Overloads and impacts can result in splitting, brooming or other damage to timber elements. Not only will these conditions immediately lower the capacity of timber sections, they will make the element more prone to attack from insects or fungus. Elements which have experienced damage may have some usable capacity remaining if the remaining portions can be protected against decay through preservative application or encasement.

Connection Deterioration

Timber elements are commonly connected to each other using steel bolts that can be susceptible to corrosion. Overloads and impacts can cause connection damage that may consist of shearing of bolts, wood damage around bolts, or wood crushing up against other elements. The severity of damage will determine the extent to which repair is possible or feasible. The ability to utilize the entire capacity of timber elements is dependent on the presence of intact connections.

Water Table

The long-term state of the water level around timber elements is highly predictive of that element’s susceptibility to decay. Partially or intermittently submerged timber elements will have ample access to both moisture and oxygen, necessary components for biological attack. Above water elements in moist environments will also be highly susceptible to decay. Permanently submerged timber elements have limited access to air and will be unlikely to decay (Hannigan 2006). The water levels at the element may vary with season and short/long-term hydrologic conditions like drought or flooding. Changes to the long-term hydrologic or streambed conditions (damming of river, dredging/channeling of stream bed, etc.) can also impact which areas are directly exposed to water or air.

Masonry Element Issues

Masonry is a common building material for older piers and abutments. Masonry elements are typically unreinforced gravity elements loaded entirely in compression. Reinforcement is possible either through original design or through retrofit, usually to resist seismic loading. The presence of reinforcement can be confirmed and mapped through NDT if it is believed to exist.

Material Properties

Many older masonry substructures will be unreinforced, or very lightly reinforced. When these masonry foundations are in good shape, the capacity of the section is rarely governed by the compressive strength of the blocks; instead it is often governed by limiting tension in bending or by the shear strength of the mortar joints.

Deterioration of Masonry Blocks

Masonry blocks can become subject to weathering forces that slowly deteriorate the blocks, and splitting forces, which have the capability of opening large cracks in a short amount of time. The weathering forces that masonry blocks are typically exposed to include abrasion, freeze-thaw cycling, impact damage, or flowing water. These forces will over time reduce the masonry to small granules that wear off the blocks (Ryan et al. 2012). The weathering process is generally slow and consistent and will continually remove material from the exterior of the blocks. Exposure to deicing salts and roadway runoff can accelerate these processes. Cracks can also open in masonry blocks and grow over time. When water can seep into the cracks and freeze, the expansive forces can cause the cracks to grow until a portion of the block breaks off.

Deterioration of Mortar Joints

Often, the most vulnerable portion of a masonry wall is the mortar joints used to hold masonry blocks together. The masonry can become exposed to freeze/thaw cycles, water intrusion, salt intrusion, or plant life intrusion. Water and freeze thaw cycles can lead to cracking of the masonry in these joints from the expansive forces during freezing. Intrusion of salts from deicing can exacerbate the condition by promoting freeze/thaw cycling, and the salts can chemically react with the mortar itself. In moist environments, plant matter can take root in the masonry. The expansion of the plant matter within masonry pores can cause cracking and further intrusion of plant life.

Observed masonry damage is often repairable, although the repair costs will be dependent on the level of deterioration. Simple cracking of exterior portions of masonry can often be remedied by cleaning and reapplying mortar to the exterior. The exterior mortar is typically removed through scraping or other means until intact mortar is reached. Mortar loss that reaches the interior of the element can be addressed through some form of grout injection that fills the cracks.

Interior Voids

Interior voids can develop inside of masonry elements though mortar erosion. Elements exposed to a hydraulic gradient may experience flowing water that can degrade the mortar and create internal voids. Shifting blocks due to settlement can cause openings on the interior of the elements as cracking opens.

Location of Rebar

Most masonry structures are designed to be unreinforced sections, but contractors may have placed steel during initial construction to provide additional strength or aid in construction. Rebar or ties may have been placed after initial construction as part of a seismic retrofit or strengthening measure. In general, rebar in masonry structures is located using the same technologies as with concrete elements, like GPR or magnetic technologies. Magnetic methods may be more effective as GPR can pick up reflections from block surfaces.

Visual/Physical Inspection

Concrete

The visual inspection of concrete substructure elements locates and documents observable cracking, spalling, efflorescence, rust staining, cold joints, or other defects. The sizes, orientation, and locations of cracking can provide preliminary indications as to their causes. If deterioration has exposed reinforcement, the presence and approximate extent of corrosion can be measured. Physical inspection of concrete is done by hammer sounding to find delaminated areas, which are essentially spalls that have not separated from the rest of the remaining concrete.

Steel

The primary basis for determining the extent of corrosion of steel elements is through visual and physical inspection. Corrosion products are removed by the inspector and the remaining steel is measured. For underwater or underground elements, the critical portions to consider will be just above the water line or ground water line, where corrosion is most likely to occur. Other issues typically considered during visual and physical inspection include connection deterioration, impacts, overloads, and cracking (especially near welds and bolts).

Timber

Visual inspection of above ground timber elements can identify visual portions of decay or deterioration. Hammer testing, when performed by a qualified inspector, can identify areas of interior deterioration with a hollow sound (Ryan et al 2012). Coring allows direct observation of the interior material of the element. The penetration resistance of the coring tool can be correlated with wood strength, although this is most effective at providing a relative comparison. Areas with much lower penetration resistance are more likely to be undergoing rot and cannot be relied upon for strength. When rot has begun to occur inside of an element, it generally will not stop until the entire element has decayed. Extensive hammer and coring testing allows identification, extent, and severity of decay. The investigation requirements beyond this will be based on the severity of the observed damage.

Masonry

Visual inspection of masonry elements identifies areas of cracking, spalling, loose masonry, abrasion, mortar loss, and plant growth. Loose mortar is typically removed during physical inspection to determine its depth.

Sampling and Testing

Concrete

The main forms of sample testing commonly applied to concrete elements are petrography, compressive strength testing, static modulus of elasticity testing, and rebar strength testing. The concrete testing is performed on samples extracted via horizontal coring from the side faces or through vertical coring from the top of the substructure. Rebar testing is performed on samples removed from the element though coring or removal of portions of rebar after cover removal. Performing rebar mapping surveys with NDT technologies such as GPR or covermeters prior to core extraction helps to avoid or sample reinforcement, as desired.

Petrography

Petrography is the examination of concrete by placing samples underneath a microscope and is a crucial tool for the initial stages of concrete element evaluation. Petrography can determine the original mix properties and the presence of freeze-thaw cracking, ASR, or DEF. ASTM Standard C856 (ASTM 2014a) provides the testing procedure for hardened concrete, and ASTM Standard C295 (ASTM 2012a) provides the testing procedure for the aggregates in concrete.

Compressive Strength Testing

Compressive strength testing is performed cores removed from either the face or internal portions of the structure, taken in accordance with ASTM Standard C42 (ASTM 2016c). Due to the variability of concrete strength and testing, it is good practice to remove and test several cores to allow for determination of the average and standard deviation of concrete compressive strengths. This sampling is covered by ASTM Standard C823 (ASTM 2012b). The MBE (AASHTO 2016) recommends using the average yield strength minus 1.65 standard deviations for the unconfined compressive strength.

Static Modulus of Elasticity Testing

Static modulus of elasticity testing can be performed on cored samples in accordance with ASTM Standard C469 (ASTM 2014b). In practice, the static modulus can also be estimated from the compressive strength in accordance with section 5.4.2.4 of the LRFD Bridge Design Specifications (AASHTO 2014).

Reinforcement Strength and Condition

When the reinforcement yield strength is not known, samples can be extracted through coring or saw-cutting. If the concrete cover has spalled or been removed, a sample can be removed from the exposed sections. Exposed rebar can be directly observed for signs of corrosion, and corroded sections can be tested to ascertain the remaining yield capacity of that section. Steel removal is ideally performed in portions where the rebar is not taking substantial load. Cutting of the rebar will lower the capacity of the section where steel removal was performed and produce a discontinuity that will require development length on either side before the capacity of the rebar can be utilized.

Vertical Coring and Wireline Geophysical Logging

Vertical coring of concrete elements allows for sampling and testing of concrete from the center of the element. Samples can be taken at many elevations, including at the interface of the foundation and soil/rock. Coreholes can be extended past the bottom of the foundation to perform sampling of the soil or rock supporting the foundation. Wireline geophysical logging of the completed corehole can provide continuous logs of important data like density, wave speed (can be correlated to stiffness), and integrity. The continuous logs obtained through geophysical logging can be correlated with samples taken at discrete elevations in the corehole. These technologies are highly beneficial for below ground elements and soil, as discussed later, although all aspects applicable to below ground elements are also applicable to above ground portions that the coring is performed through.

Steel

Removal of a representative steel section from the foundation allows for testing of the structural properties, assuming steel removal will not impact the capacity or stability of the substructure. After removal, the steel element(s) can be subjected to Brinell hardness testing, Charpy impact testing, chemical analysis, and tensile testing. The MBE (2016) recommends using the average yield strength obtained from testing minus 1.65 standard deviations.

Timber

Boring uses a steel drill bit, either hand cranked or machine powered, which is advanced into the timber element. The rate of penetration, the torque required to advance the bit, and observations of the tailings are used to infer information about the quality of the wood. This testing allows identification of areas that have undergone decay. Drill resistance uses a small diameter (1.5mm - 3mm) hole to determine density end detect decay of timber. This hole is small enough to have only negligible structural effects on the remaining cross-section of wood and may be sealed to prevent access for agents of decay. Coring uses a hollow stem drill bit or auger that allows removal of a timber core. The hollow bit can be opened after removal from the element. The removed core can then be directly observed for decay, voids, or other issues. The removed cores can be cultured to detect whether there are biological agents (fungi) present.

Masonry

Vertical or horizontal coring and sample removal can be used to identify the block strength, similar to comprehensive strength of concrete. Compression testing and static modulus of elasticity testing can then be performed on the removed cores. ASTM Standard C1532 (ASTM 2012d) specifies the procedure for removing masonry cores, while ASTM 1587M (ASTM 2015b) specifies the procedure for preparing the masonry samples for compressive testing. An outline of the procedure, how to choose samples, and how to interpret the results is available in the National Concrete Masonry Association *Evaluating the Compressive Strength of Masonry* (NCMA 2011).

Summary of NDT Technologies

Concrete and Masonry Elements

Table 12 presents a summary of non-destructive testing (NDT) technologies that can be used for concrete elements. Many of these same technologies can be applied to masonry structures as well. Technologies like SE/IR, US and PS may be able to identify cracking and internal voids in masonry, although the boundaries between the masonry blocks and the mortar can limit how effectively the results can be analyzed.

Table 12. NDT methods for concrete elements and their use

NDT Method	Issues Investigated
Ground Penetrating Radar	Rebar layout, voids, cover depth
Ultrasonic Pulse Velocity and tomography	Location of voids, weak zones, honeycombing, and cracks
Infrared Thermography	Location of voids and delaminations
Electrical Resistivity (ER)	Presence of water, chlorides, and salts
Radiography	Location of voids and condition of tendons and strands
Rebound Hammer	Surface strength of concrete
Impact Echo/Ultraseismic/Parallel Seismic	Location of defects and voids in piles

Ground Penetrating Radar (GPR)

Ground Penetrating Radar (GPR) uses two antennas, one that transmits and another that receives the reflected electromagnetic waves. GPR is used to assess subsurface flaws and to image embedded reinforcement or tendons. A common application of GPR is on concrete bridge decks to determine the rebar cover and evaluate rebar corrosion as well as the thickness and debonding of asphalt overlay. High frequency GPR antenna (1 GHz and higher) is most effective when used to detect anomalies within 12 inches of the test surface; lower frequency systems can penetrate deeper. For exposed bridge substructure elements, GPR can be used to detect voids, the location of rebar near the surface, and the location of post tensioning ducts. GPR signal is influenced by the corrosion process; although it is not a reliable predictor of delaminated areas. The use of GPR to find rebar or post-tensioning is discussed in the capacity evaluation section.

Ultrasonic Pulse Velocity (UPV) and Tomography

Ultrasonic pulse velocity measurements and ultrasonic tomography can also be used to detect and image the location of voids, cracking, or honeycombing within a concrete element (Bungey et al. 2006). These methods use a transmitting transducer to generate ultrasonic pulse that is then received by an accelerometer on the other side of the element. Comparing the relative speed between various ultrasonic pulse velocity measurements, cracks and voids can be located and their size estimated from the difference in wave speed, as waves will need to pass around cracks. Ultrasonic pulse velocity measurements do not generally give direct measurement of concrete strength and modulus but provide a good representation of how variable these quantities are. When correlated with core testing, a good picture of the variability of concrete strength can be found. Ultrasonic tomography can be employed, when utilized at multiple angles, to provide a 3D tomographic mapping of the internal configuration of the element.

Infrared Thermography

Infrared thermography is another method of detecting defects within concrete elements. This technique relies on measuring the heat flow through concrete and detecting areas with different heat conduction than the rest of the element. The primary usage of this technique has been to determine if delamination of concrete has occurred, as this would directly impact the heat flow characteristics of the concrete near the surface. IR measures the amount of infrared energy emitted by an object to calculate temperature for assessing deterioration, surface and subsurface flaws, and moisture intrusion.

Electrical Resistivity (ER)

Electrical resistivity (ER) is used to assess concrete's susceptibility to corrosion, moisture, and chloride penetration. ER can potentially identify corrosive environments in concrete created by the presence of water, chlorides, salts, and other corrosive substances. Damaged and cracked areas can form pathways for fluid and ion flow, resulting in higher concrete electrical conductivity (or lower electrical resistivity) sections in the data.

Radiography

Radiography uses high-energy x-rays directed at elements with a special photographic film placed on the back side to produce a photograph like representation of the internal structure of the tested member. Voids or inconsistencies within the element will then be exposed onto the x-ray film. Radiography can be used to determine the condition of tendons and strands, detect voids in cast-in-place or prestressed concrete girders and detect grout voids.

Rebound Hammer

For simple surface evaluation, a rebound hammer can be employed following ASTM Standard C805 (ASTM 2013a). This investigation can provide a measure of the surface hardness of concrete that can be correlated to strength. This method is not typically an accurate measure of absolute strength but can be useful for comparing the relative strength of the surface concrete.

Impact Echo (IE)

IE utilizes impact generated stress waves to assess subsurface flaws and material thickness. IE can be used to determine location and extent of hidden flaws including cracks, delaminations, voids, honeycombing, and debonding. IE has been successfully used to detect and locate voids in the grout of bonded post-tensioning tendons in bridge decks and girders. IE is also used to determine thickness of slabs, decks, walls, or other plate structures. IE only requires access to one side of structure. IE testing is sometime performed along with surface wave analysis for evaluating material properties (elastic moduli) of concrete.

Spectral Analysis of Surface Waves (SASW)

Spectral analysis of surface waves (SASW) can be used to determine the stiffness of concrete and masonry elements and to locate voids and delaminations (Wightman et al. 2003). This method relies on measuring the propagation of surface (Rayleigh) waves through the structure. The waves are initially introduced into the structure with a small hammer tap, and the waves are measured with multiple accelerometers placed in a line with the hammer strike. Rayleigh waves have frequency dependent velocities and wavelengths, so this technology can be used to identify the depth of voids and delaminations. Further information on SASW can be found in Wightman et al. (2003).

Sonic Echo (SE), Impulse Response (IR), Bending Wave, and Ultraseismic (US)

These technologies are discussed in greater detail in the section on below ground elements, as they are most advantageous for slender elements where only the tops of the elements are observable and/or accessible. Primarily, these technologies are used to identify cracking, voids, and element length; quantities that may be readily observable in above ground elements. Nevertheless, these technologies are perfectly viable for above ground elements, and they are often employed on elements with both above and below ground portions. Ultraseismic (US) is particularly well suited for determining the depth and integrity of masonry piers and abutments that may be primarily above ground.

Steel

Dye Penetrant Testing (PT)

Dye penetrant testing can be performed to identify hard to notice cracking and surface flaws in members. This method requires the inspector to clean the surface of the area being inspected, apply the dye, and then remove excess dye on the surface. Dye that has seeped into cracks can then be drawn out with a special "developer" material due to capillary action, showing the locations and sizes of cracks. This method does not require extensive training to use.

Magnetic Particle Testing (MT)

Magnetic particle testing can detect surface breaking cracks on ferromagnetic materials such as steel girders and steel truss members. When a specimen is subjected to a magnetic field, the presence of a defect will cause local distortions in the magnetic field around the defect area known as magnetic flux leakage. If fine particles of magnetic material are placed on the specimen in the presence of the magnet field, they will be attracted to a defect area due to the presence of the magnetic flux leakage around the defect.

Eddy Current Testing (ECT)

Eddy current method utilizes electromagnetic induction to assess surface flaws, material thickness, and coating thickness and is typically used on metals with painted or untreated surfaces. Material properties and discontinuities, such as cracks, disturb the eddy current trajectories, affecting the magnitude and phase of the induced current. This method does not work with galvanized metals and only provides information on the presence of cracking.

Ultrasonic Testing (UT) and Phased Array Ultrasonic Testing (PAUT)

UT utilizes high frequency sound energy to assess flaws (surface and subsurface) and dimensional measurements and is typically used on metals with untreated or cleaned surfaces. UT is used to detect cracks in steel members and weld, pins or anchor bolts, and can also assess thickness of steel members for determining geometry or assessing section loss. Phased array ultrasonic testing (PAUT) is performed with transducers made from many individual sensor elements creating an ultrasonic beam that can be steered in different directions or focused at different depths. PAUT can image the presence of subsurface flaws from poor initial build quality or from stress and fatigue cracking. PAUT can also be used for in-depth inspection of weld connection and hanger pin defects.

Acoustic Emission (AE)

Acoustic emission testing is commonly used to monitor crack growth in steel members. As steel deforms due to applied loads, cracks and imperfections will produce pressure waves. These pressure waves, referred to as acoustic emissions, are generated by crack growth or plastic deformation (Hopwood and Prine 1987), and are typically in the ultrasonic range, with frequencies of 100 kHz to 1000 kHz (Hopwood and Prine 1987). The acoustic emissions by themselves do not indicate the severity of the cracks found. For acoustic emission testing to provide results, the cracks or discontinuities must be actively growing, as that is what produces the emissions.

Timber Elements

Stress waves

Stress wave methods use an impact hammer or ultrasonic transducers to generate a stress wave that is transmitted through a timber member or is reflected from internal flaws or boundaries. Simplest methods for utilizing stress waves is measuring time of flight of stress waves between a predetermined length (accessible boundaries) of a timber member. Stress wave travel slower in decayed wood than sound wood. Knowing distance, the stress wave velocity is calculated which enables determination of modulus of elasticity (MOE) and estimation of strength properties. Spectral analysis of surface waves (SASW) measures the dispersion of Rayleigh waves to identify shear stiffness and locate voids and discontinuities. Since the wavelength of Rayleigh waves is frequency dependent, it can identify the depth of voids and discontinuities within timber elements.

Electric Moisture Meters

The electric moisture meters are used to determine the moisture content (MC) in wood. The electric moisture meters measure electric conductance (resistance) or dialectic properties that correlate with moisture content less than 30 percent. A more advanced resistivity probe that uses a pair of equidistance current and potential probes provides electrical resistivity as a function of depth.

Ground Penetrating Radar (GPR)

GPR involves the propagation of electromagnetic waves at frequencies typically between 100 MHz to 2 GHz. GPR is useful for detecting internal defects such as fungal decay, rotting, piping, and cracking in dry members and potentially lateral changes in moisture content (MC). GPR surveys are fast and only require one sided access. Microwave imaging is similar to GPR but can obtain higher image resolution. Like GPR, microwave imaging can measure internal defects and relative MC, and the higher resolution can potentially determine slope-of-grain, density of knots, and specific gravity of timber members.

STRUCTURAL INTEGRITY OF BELOW GROUND ELEMENTS

Unlike above ground elements, portions of the foundation that are below the ground surface cannot be directly observed without excavation. Therefore, evaluation of these members relies more heavily on NDT methods capable of indirectly detecting the geometry of buried elements as well as any damage to buried elements.

Foundation Depth

Determining the foundation depth is crucial to determining the susceptibility of a foundation to scour. Foundations with unknown depths, termed “unknown foundations” (Schaefer and Jalinoos 2013), are not typically candidates for reuse, as significant investigation may be needed as part of the reuse investigation. Known foundations being considered for reuse still may have missing critical documents such as design plans, material properties, as-built plans, boring logs, or installation logs. Since soil profiles and conditions can be so variable, the as-built structure may have key differences with the design plans. Even when as-built plans are available, it is good practice to confirm their accuracy with limited testing. The risks associated with reusing these foundations can be mitigated though the use of technologies that help determine/confirm the foundation depth. Technologies available for determining the depth of a foundation include: corehole logging, surface NDT methods (SE/IR/US), borehole NDT methods (PS/IF), and exposure of elements with test pits. Table 13 shows which technologies are applicable for various foundation types.

Table 13. Investigation type available for determining concrete element depth

Foundation Type	Methods to Determine Depth		
	Test Pits	NDT	Coring
Footings	×	×	×
Drilled Shafts		×	×
Driven Piles		×	

Test Pits

The depth of shallow foundations and footings can be determined from test pits, where an excavation is performed to the bottom of the footing. This allows direct observation of the footing depth, and observation of the footing condition. This method is only possible for footings where the excavation can be performed without undermining the foundation. Drilled shafts and driven piles are too far below ground surface for this method to expose the entire foundation depth, however test pits are commonly performed to view the condition and locations of piles or shafts. Coring, as discussed for concrete and masonry elements, can be used to identify the depth of the foundation by sampling the interface of foundation and soil/rock. Various NDT methods, as discussed below, can determine the foundation depth.

Damage in Foundation Elements

Damage to Driven Piles

Driven concrete piles can become damaged during installation, especially if the hammer energy was inappropriate for the material driven through or if piles were driven hard past boulders or obstructions. When soil exploration data indicates that piles were not driven through particularly difficult (excessively hard or soft) material, it may be a reasonable assumption that the history of adequate performance from the foundation is evidence the piles were not damaged during driving. However, it is possible to have had adequate performance from a pile group with a damaged pile. The most effective technique for managing this risk is through driving logs that document the entire installation. In absence of these logs, various NDT technologies can identify damage in underground piles, though it may be impractical to perform this testing on every pile.

Construction Defects of Drilled Shaft Foundations

Construction defects of drilled shafts include honeycombing, cold joints (lift lines), voids, soil intrusions, caving, bulbing, poor-quality concrete (weak zones), and soft bottom (Jalinoos et al. 2005). The presence of construction defects can lower the capacity of the section and become focal points for corrosion or other deterioration. Honeycombing, cold joints, and voids are similar to issues that can exist for above ground elements and are discussed in that section. Soil intrusions, caving (decrease in diameter), and bulbing (increase in diameter) occur in drilled shafts due to side wall issues during construction. Soft bottoms are formed at the bottom of the shaft due to improper bottom cleaning or the mixing of bentonite mud with concrete under the wet construction method. Most agencies require crosshole sonic logging (CSL) or other NDT methods on new drilled shafts for quality assurance (Jalinoos et al. 2005), although drilled shafts being considered for reuse may not have been subjected to this level of QA.

Surface NDT Methods

Surface NDT methods use measurements taken from the above ground portions of the substructure to infer properties and conditions of below ground portions. In general, they function by inducing wave at the top of the elements with a hammer or impact device and using accelerometers to record the wave propagation through the substructure element. The methods do not generally require excavation or boreholes to perform. A summary of various surface NDT methods can be found in Wightman et al. (2003).

Sonic Echo (SE) and Impulse Response (IR)

Sonic Echo (SE), and Impulse Response (IR) refer to two methods of performing pile integrity testing (PIT), also known as low-strain impact integrity testing. The procedure for low-strain impact integrity testing is given by ASTM Standard D5882-16 (ASTM 2016a), and generally involves inducing a compressional wave into the element and using an accelerometer to listen for echo delay to locate cracks, voids, anomalies, and the bottom of the element. Sonic echo (SE) testing relies on time domain analysis of the reflected waves, while impulse response (IR) testing relies on frequency domain analysis. Elements with high wave speed uncertainty will similarly calculate depth with increased uncertainty.

Sonic echo (SE) testing is commonly performed on timber elements to locate voids and determine foundation depth. SE testing on timber piles can be performed by inserting two probes into the wood near the top of the element; one to create an impulse and one to record wave reflections. This method is considered effective for elements made of Douglas Fir or Western Red Cedar but does not work well with Southern Pine (Ryan et al. 2012). Similarly, IR can use the same setup to estimate the extent of decay and the in-situ strength of the timber elements.

Bending Wave Methods

Another variation of the stress wave method, the bending wave test uses the reflection of dispersive flexural waves, rather than compressive waves, to determine the unknown depth of the foundation or large defects. Both approaches use one or two receivers placed near the point of the hammer strike to listen for the response. A summary of bending wave methods can be found in Wightman et al. (2003).

Ultraseismic (US) Testing

Ultraseismic testing (US) is similar to SE/IR testing, except multiple accelerometers are placed along the side of the element (Jalinoos, et al. 1996, Jalinoos et al. 2017). Vertical and horizontally polarized accelerometers are placed at discrete locations along the side of the pier for vertical profiling, and along the top of the wall foundation for horizontal profiling. The hammer (impulsive) strike or swept frequency (chirp) type sources are recorded by the receiver array. The increase in time of arrival for the wave front (moveout) allows the wave speed to be directly measured. Echoes from large defective zones or pile tips can be measured with high confidence by removing sources of uncertainty with SE/IR testing.

Seismic Wave Reflection Survey

Seismic wave reflection surveys are performed by setting up a grid of geophones on one side of foundation while inputting an excitation into the ground on the other side. Since waves will reflect off the bottoms of the foundation elements, the depth of these elements can be determined by locating where the reflection reaches the ground surface.

Borehole Investigation Methods

Borehole investigation methods use boreholes installed near existing foundations to infer information about the depth, location, and integrity of foundation elements. In general, these technologies consist of lowering logging equipment down completed boreholes and exciting the structure under investigation with an impulse or magnetic energy, while recording the response with sensors in the borehole. Some borehole investigation methods lower both a transmitter and receiver down the borehole to locate elements using wave reflections. Borehole investigation methods are highly effective at determining the depth of installed elements and can identify discontinuities or break in existing elements. These methods are discussed in greater detail with visual aids in Wightman et al. (2003).

Parallel Seismic (PS) Testing

Parallel seismic (PS) testing is a borehole-based method that can determine gross structural integrity and depth of driven piles or drilled shafts. The PS method involves drilling a borehole in the proximity (3-4 ft, ~1 m) to an existing pile or drilled shaft. A hydrophone or geophone (receiver) is lowered in the borehole. The structure is then struck with a high-energy impulse while the receiver records the acoustic response at depth within the borehole. This procedure is repeated multiple times with the hydro/geophone located at various depths within the borehole. Since the energy waves will propagate through the pile at a different speed than through the soil, the location of the bottom of the element, discontinuities, or breaks in a pile can be determined through analysis of the time of wave arrival at various depths. When the receiver is deeper than the foundation, increases in depth will cause the time of arrival to increase at a different rate than when the receiver is alongside the foundation.

Induction Field (IF) Testing

Induction field testing uses a magnetic sensor lowered into completed boreholes to detect the presence of metal objects, like steel or reinforced concrete (Wightman et al. 2003). A current is impressed in the metal pile, and the sensor is then placed at various depths while the pile is charged. Since the magnetic field drops off rapidly as the sensor is moved away from the pile, the bottom of the pile can be located.

Borehole Radar and Sonic

Borehole radar and sonic methods use a device lowered into a completed borehole that emits microwaves or acoustic waves. A receiver is placed into the borehole alongside the emitter to detect the reflection of these waves off nearby foundation elements. By performing this test at various depths, the depth of the bottom of the foundation can be determined.

Vertical Coring and Wireline Geophysical Logging

Vertical coring can be performed on existing foundations to directly measure foundation depth, integrity, material properties, and soil properties. Inside the completed coreholes, logging probes can be lowered to perform wireline logging and estimate the material properties and conditions surrounding the completed borehole.

Coring Procedure

Bridge piers, abutments, caissons, and drilled shafts can be cored with a core barrel attached to a standard geotechnical drill rig. The coring procedure allows relatively undisturbed samples to be removed from the center of the foundation element. Samples obtained during coring can be subjected to compression testing, petrography, or visual classification (e.g., providing a rock quality designation (RQD) like standard rock coring). Cores drilled vertically through concrete elements can be continued into underlying soil or rock, providing a sample of the concrete/rock interface, and directly sampling the material supporting the foundation. Standard sampling and testing of the soil and rock can be performed from this corehole like a typical borehole. Coreholes are often performed directly from the bridge deck by coring through the bridge deck, past the girders, and into the top of the substructure. This requires lane closure during drilling, but also allows for easy and convenient access. This process allows for retrieval of inner portions of concrete, the boundary between concrete and soil/rock, and the underlying soil/rock. Coreholes that cannot retain water are an indication that highly porous zones exist or that there are cracks allowing the water to escape.

Wireline Logging

Wireline logging is performed by lowering logging probes into a completed corehole. It is used to measure material properties, to locate defects and cracking, and to identify local zones of weaker concrete. Density logging, most commonly used for rock foundations, can be used to provide a continuous log of the concrete density. Full Waveform Sonic (FWS) Logging can determine the P-wave, S-wave, and Stoneley wave velocities of the concrete. Caliper logging provides a continuous measure of the corehole diameter. Sections with increased diameter may be caused by voids, inclusions, or weak zones within the element. After completion of a corehole, a televiewer can be lowered down the open hole to provide and oriented view of the sides of the corehole. This oriented digital map provides the locations of cracks and voids that intersect the complete corehole. Both acoustic and optical televiewer methods exist, with acoustic using a sonic echo to map the sides, and optical using a camera. Optical televiewers require that corehole be dry or filled with clear water, while acoustic televiewers can be used in muddy water.

Crosshole Sonic Logging

Crosshole Sonic Logging (CSL) is typically performed on newly constructed drilled shafts in accordance with ASTM Standard D6760-16 (ASTM 2016b). This technology works by generating sonic waves in the range of 30 kHz to 40 kHz and measuring the speed of the sonic waves (Wightman et al. 2003). Areas of cracked or weakened concrete will exhibit slower wave speeds than intact concrete. This testing is used to ensure that the constructed shaft has no voids or discontinuities, and some indication of the concrete properties can be estimated from the wave speed of the concrete. Other testing, such as gamma density logging or temperature logging can also be performed by owner agencies (Jalinoos et al. 2005). While many State and local agencies require NDT testing to be performed on new foundations, the testing may not have been performed on existing foundations, or the data may not exist. This technology requires that access tubes be installed prior to concrete curing, although CSL can be performed on existing foundations where these tubes are installed.

Summary of Technologies

Table 14 provides a listing of various wireline logging technologies, their uses, and notes on the technology.

Table 14. Main corehole logging technologies and their uses

Logging Technology	Measured Parameters	Notes
Optical televiewer (OTV)	Digitally images the inside of corehole wall using optical camera. Records an oriented 360°unwrapped and 3D image of the corehole wall or a "digital core."	Ideal for air filled coreholes. For water-filled holes, clear water is required. It can pick the orientation of micro-cracks in structural elements or bedrock
Acoustic televiewer (ATV)	Oriented images inside of the fluid filled corehole using acoustic transducer. Provides similar imagery as Optical televiewer.	Requires fluid filled holes and works in muddy (unclear) water.
Caliper logs (mechanical and acoustic)	Measure the corehole diameter and any change due to voids or washout zones in soil or bedrock.	Determines the change in the diameter of the corehole wall and the depth of voids
Full waveform sonic (FWS) log	Measures compressional (p), shear (s) Stoneley, and tube wave arrivals and amplitude.	Along with density logs, elastic (mechanical) properties logs can be derived to display shear, bulk, Young's moduli and Poisson ratio as a function of depth.
Density log (compensated and 4-pi)	Determines material density.	Compensated density measures density values as a function of depth. 4-pi density is mostly used to detect defects.
Electrical resistivity logs	Determines electrical resistivity of material at different radius of investigation as well as single point resistance (SPR) and spontaneous potential (SP).	Can identify areas of high conductivity in concrete/masonry, and possibly rebar corrosion in rebars in concrete
Electromagnetic Induction logs	Measure electromagnetic conductivity at typically 2 radii of investigation	Can measure areas of high conductivity and steel.
Thermal neutron log	Measures the amount of hydrogen atoms in a formation.	Its main use is in the determination of porosity or presence of moisture.
Gamma log	Measures the amount of gamma radiation produced mainly by isotopes of potassium, thorium, and uranium.	Can identify differing concrete mix or concrete deterioration.

Potential Uses of Geophysical Logging

Geophysical logging through wireline logging, crosshole sonic logging, and single hole sonic logging can be used to improve knowledge about below ground structural components. Some of the potential uses of geophysical logging are listed below.

- Crack Mapping Logs – using optical televiewer (OTV), acoustic televiewer (ATV), and mechanical/acoustic caliper logs for mapping cracks, in-place condition, or imaging voids. The OTV/ATV can detect microcracks and determine their orientation. The OTV/ATV logs can also image voids with their depth computed by mechanical/acoustic caliper logs.

- Physical (Elastic) Property Logs – using a combination of full waveform sonic (FWS) and compensated density logs to produce dynamic moduli (Young, shear, and bulk moduli) logs and Poisson Ratio logs continuously as a function of depth (typically 1-2 in (2.5-5 cm)).
- Material Property Logs
    - Natural gamma and spectral gamma to assess different concrete mixes and properties. In concrete, the ^{40}K gamma counts mostly originate from the Portland cement. In some cases, gamma counts can be due to aggregate if crushed granite or trap rock were used, but this is not common. Elevated ^{40}K gamma counts can also infer potential (but not positively identify) concrete deterioration due to alkali silica reaction (ASR) or delayed ettringite formation (DEF). Petrographic analysis of cored samples can be used for final verification of suspected zones.
    - Thermal neutron logs for presence of moisture important for assessing potential for corrosion or other concrete deterioration.
- Corrosion logs – resistivity and electromagnetic induction logs to evaluate potential for rebar corrosion.
- Structural Integrity logs – detects voids and defective construction zones such as soil intrusion, bulbing, soil intrusion, necking, honeycombing, cold joints, poor quality concrete and soft bottom.
    - 4-pi gamma-gamma density log – detects defects about 3-4 (7.6-10 cm) in from the corehole wall.
    - FWS sonic – potentially detects defects using the reflection of compressional wave in the sonic record or changes in tube wave characteristics. FWS is a more advanced form of single hole sonic logging (SSL).
    - Borehole radar – detects reflection of radar data from defects and voids in stone masonry elements. In heavily reinforced concrete, borehole radar data can be unusable due to strong reflection from steel masking the defect signals.
    - Ultraseismic vertical profiling (similar to downhole seismic in geophysics) uses a seismic source on top of the foundation and a hydrophone (or geophone) string in the corehole to record the seismic response. Ultraseismic logs indicate large defect zones along the length of the foundation.
    - Cross-corehole tomography - requires two or more coreholes for two-dimensional volumetric imaging between coreholes pairs. Crosshole sonic logging (CSL) equipment can be used if the corehole separation is less than 10 ft (3 m); otherwise, higher energy seismic equipment consisting of downhole sources and hydrophone strings may be deployed.

When coreholes are extended through the foundation element into the underlying soil or rock formation, corehole logging can also be used to evaluate geotechnical properties directly below the foundation tip. This information, supplemented with soil properties from a borehole drilled close to the foundation, can evaluate geotechnical integrity. Thus, wireline logging is a key technology for evaluating structural and geotechnical integrity of the substructure elements in this application.

GEOTECHNICAL SYSTEM INTEGRITY

The integrity assessment of the geotechnical system primarily seeks to characterize the performance of the existing foundation, the presence of scour, and the in-situ soil properties at the time of reuse consideration.

Existing Foundation Conditions

Pier and Abutment Movement

Settlement, translation, and rotation limits often govern the design of foundations. One of the primary criteria governing the design of pier and abutment foundations is the limit on allowable settlement, translation, or rotation (Davis et al. 2018[a]). The existing foundation will have undergone movement during its original service life. Estimations of these movements can be made from observations, surveys, or other measurements. If a continuous monitoring program for the settlement and translation of the piers and abutments is in place, the total amount of movement, the rate of that movement, and how that rate is changing with time can be measured. Having an estimate of the total movement experienced during the existing service life of the foundation provides an upper limit to the amount of movement that will occur during reuse, assuming loads are roughly equivalent. The rebound movement during unloading can be monitored to provide additional confidence to the magnitude expected, as discussed in greater detail in Chapter 6.

Beyond vertical movement, abutments and wingwalls for reused foundations can be subjected to lateral movement. Signs of bowing, cracking, or excessive deformation can indicate that the earth support wall is failing to retain the soil behind it.

Bedrock Depth

Determining the elevation of bedrock on the site is frequently an important aspect of the geotechnical evaluation. The elevation is a crucial parameter when assessing scour vulnerability, capacity assessment, settlement predictions, and slope stability. Existing soil exploration data can identify the location of bedrock at discrete locations where the original boreholes were performed. The information can be supplemented with additional boreholes, although boreholes are only capable of providing the bedrock elevations. When bedrock elevation is highly variable, or it is crucial to map the surface of bedrock, surface and borehole geophysical methods can be employed that provide 3D contours or 2D slices of bedrock depth.

Drainage

Foundations that retain soil are frequently constructed with weep-holes, backfilled with crushed stone, or provided other drainage paths that prevent the buildup of hydraulic head between the groundwater on either side of the wall. Existing structures may have inadequate drainage or clogged drainage paths that allow the buildup of hydraulic head behind the wall. When a foundation is being evaluated for potential reuse, various approaches can be used to evaluate the drainage performance of the retention structure. A simple approach may involve observing if water is exiting weep-holes during or after a rain event (although this may be complicated if the wall is directly exposed to rain). Another approach would be to install observation wells near to the retaining structure that can be sampled during/after rain events to determine if the groundwater level behind the wall is changing. If needed, it may be possible to excavate portions of the retained soil to observe gradation, density, and porosity. If drainage issues are identified, corrective actions can be taken, or the foundation can be designed to withstand the hydraulic loads.

Previous Scour

Bridge foundations that are being considered for reuse and are in rivers, estuaries, or other bodies of water may have experienced scour during their initial service life. The evaluation of this existing scour can provide valuable information that can inform the reuse evaluation and reduce uncertainty associated with the foundation. When evaluating existing scour at bridge foundations, it may be helpful to consider both the magnitude of the observed scour and the extent of prior flooding. The extent of prior scour in comparison with estimations of the expected scour can help inform design choices used to estimate scour potential. For example, Hunt (2009) documents several case studies where measurements of scour were performed on bridges where scour was predicted through analytical equations. It was found that large deviations from the predicted scour are possible, often due to assumptions made about stream and soil properties. By comparing the observed scour with the predicted scour, some uncertainty associated with these calculations can be reduced. It is common for local scour to fill back in with new material, possibly in a matter of two to three months for sandy material and up to six months for fine grained material (Ghosn et al. 2003). This refilled material is often less dense than the original scoured material and therefore provides less geotechnical resistance. While visual or acoustic methods may not identify the presence of refill material, methods like rod sounding, probing, or additional borings can identify the density of the material. Various methods, as described in table 15 can be employed to estimate the amount of scour currently present on a bridge site. The evaluation of current scour can provide meaningful insight on the scour hazards present at the bridge. Potential future scour will impact the capacity of the foundation, as discussed in Chapter 6.

Geohazard Evaluation

The soil exploration program also typically investigates potential geohazards, including:

- Scour
- Seismic hazards
- Corrosive soils
- Karst formations

Table 15. Scour and pier inspection methods

Method	Description	Advantages	Limitations
Rod sounding	Metal rod used to probe ground surface around pier.	Inexpensive, effective at identifying scoured soil.	Requires wadable water or easy access to foundation, may not always distinguish infill soil from virgin soil.
Visual inspection with diver	Diver and team visually and physically inspect bridge pier	Can inspect entire pier during inspection, provide photos in clear water, check for loose pier material	Labor intensive. Requires specially trained divers. Drivers may not be able to distinguish between infill and virgin soil
GPR	GPR system adapted to map underwater ground surface	Fast, relatively inexpensive, can distinguish infill soil from virgin soil.	Requires water to be less than 25' deep. Less effective in saltwater or conductive soils (clays).
Acoustic Imaging, Subbottom Profiling & Fathometers	Sonar technology for horizontal or vertical imaging. Sub-bottom profiling can image geological section and paleo scour.	Can provide imagery of pier and ground surface, even in murky water.	Lower resolution than visual imagery.

Scour Vulnerability

HEC-18 (Arneson et al. 2012) addresses the primary concerns and methods for evaluating scour at bridges. Existing foundations were not necessarily constructed in accordance with these standards, so when applicable, it is good practice to confirm potential scour depths and assess the vulnerability of the structure to scour. Important considerations when assessing scour include the foundation depth, grain size distribution and properties of in-situ soil, and foundation performance under scour conditions. Assessing the previous scour observed at the bridge, the streamflow conditions that caused this scour, and the substructure behavior post-scour provide valuable insight into the resilience of the bridge to hydraulic hazards.

Seismic Hazards

Geotechnical evaluation of seismic hazards generally involves two distinct activities: identifying the site amplification of ground motions and identifying individual hazards (liquefaction, settlement, slope failure, fault rupture) that can occur at the site. Site amplification and geohazard analysis can be based on data obtained from routine subsurface investigation, such as standard penetration testing (SPT), sampling of soils to determine gradation and geotechnical properties, and determining the long-term groundwater fluctuations. Advanced exploration techniques, such as cone penetration testing (CPT) or flat-plate dilatometers (DMT) can provide more thorough coverage of the subsurface when considering these issues. Downhole geophysical wireline density logging can directly measure density as a function of depth. Seismic surface wave methods (SASW/MASW) and downhole seismic testing can measure shear wave velocities in soil that can be correlated to the density and can be used to map subsurface density. A brief description of specific seismic geohazards that can impact a bridge site include:

Site Amplification: Loose soils surrounding a foundation can amplify bedrock ground motions and cause greater spectral accelerations at a bridge site. Generally, the site amplification is defined by the type of material present, their shear wave velocity and/or the relative density. Evaluation of site amplification and seismic hazard spectra can be carried out by following the provisions in the LRFD Bridge Design Specifications (AASHTO 2014), AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2011) with interim revisions or local seismic design specifications, such as New York City Department of Transportation Seismic Design Guidelines for Bridges in Downstate Region (NYCDOT 2016).

Liquefaction: Liquefaction occurs as excess pore pressures develop in saturated or nearly saturated soils during an earthquake. The excess pore pressures cause a drastic reduction in shear strength and can cause the soil to behave more like a liquid than a solid. The resulting hazards include flow slides, lateral spread, reduction in bearing capacity, ground settlement, and increased pressure on retaining walls (Buckle et al. 2006). The Southern California Earthquake Center (SCEC, 1999) and Buckle et al. (2006) provide in-depth guidelines for assessing and evaluating liquefaction.

Settlement: Post-earthquake soil settlement may occur as a result of liquefaction or densification of soil during seismic loading. Unlike liquefaction, earthquake-induced soil settlements can occur in soil that is above the groundwater table. Soil settlements observed at a bridge site can lead to differential settlements that can distress or damage bridge components. Sites underlain by stiff clays/silts, very dense sands, or materials with a geologic age greater than 11,000 years are less susceptible to seismic soil settlement (ATC/MCEER 2002). Important criteria to consider during settlement analysis are; the thickness of the layer prone to settlement, the density of the layer, the gradation of the soil, and the magnitude of the design earthquake.

Slope and Retention Wall Failure: The primary concerns with retention structures include overturning/instability, bearing failures, structural overloads, and damage to nearby structures resulting from wall movement. Important concerns for retaining walls during this evaluation include: wall geometry, liquefaction potential, anchor capacity (when applicable), soil properties behind the wall and in front of wall toe, groundwater position, wall drainage, and footing bearing capacity. Slope failures can result from liquefaction of the slope, rockfalls, or reactivation of previous landslides (Power et al. 2006). Slopes may pose a hazard to bridges even when the bridge is not directly over the slope, and landslide can collide with the bridge or remove supporting material.

Surface Fault Rupture: Surface fault rupture occurs along the ground surface, usually directly over or very near the active bedrock fault that ruptures during the design earthquake. This hazard is relatively rare, as a bridge would need to be located directly over an active fault for this to occur. However, surface fault rupture is an especially troublesome issue to deal with, as it can have catastrophic effects and be cost-prohibitive to mitigate.

Flooding: Flooding can occur during a seismic event due to tsunamis, seiche (sloshing of lakes and bays), waves from landslides into a body of water, damming of rivers due to uplift, or dam failure (Buckle et al. 2006). These events can cause scouring of soil, water loading on the structure, or overtopping of the bridge. In general, these hazards are evaluated on a case by case basis based on the identified threats.

Overall, the vulnerability of existing bridge foundations being considered for reuse can be assessed using applicable codes such as current versions of LRFD Bridge Design Specifications (AASHTO 2014) and other state-of-the-practice resources available and design specifications developed for the specific bridge project.

Corrosivity

Soil corrosivity is strongly linked to soil resistivity, which can be measured with borehole geophysical logging. Other factors include soil pH, chloride content, and sulfate content, all of which can be directly measured from borehole samples. Various geophysical technologies, like resistivity and conductivity measurements, can be used to estimate the presence of ions and contaminant plumes in soil. The presence of corrosive soils often increases the amount of integrity investigation required for subgrade elements.

Karst Formations

Karstic bedrock formations can lead to highly variable rock depths that do not necessarily conform to design plans. Traditional boring and sampling may not find large holes in the bedrock formation that can be filled with loose infill type material. If these holes are under or near the bridge footprint, they can expose the foundation to seismic vulnerabilities and sinkhole potential. Electrical resistivity testing performed in coreholes or boreholes allows for imaging of the bedrock surface through resistivity differences with the overlying soil.

Original Soil Data

Original soil data is often available in the form of boring logs and standard penetration test (SPT) blow counts. Existing foundations may have been constructed with a small amount of soil sampling and testing (by modern standards), and data taken at the original time of testing may not have been preserved. Evaluation of existing geotechnical data is an important component of the desk study, where additional evaluation needs are determined.

One important distinction to make with old data is with the equipment used to obtain samples. Original SPT testing was probably not performed with an automatic hammer, and it is likely that the results are more variable and have a lower efficiency factor than those using new SPT testing. Other common data that might be available from the original installation includes grain size distribution (sieve analysis) and relative density testing performed on backfills. Even for bridges where density testing was not performed, specifications may supply the compaction requirements in terms of the percentage of the modified proctor dry density.

New Soil Exploration

On the case studies observed, it has been standard practice to perform additional borings and soil exploration to confirm the conditions on site. Even when original borings from the time of installation are available, further exploration is typically conducted to confirm that data from past borings is consistent with newly acquired data. More advanced testing, such as flat-plate dilatometers (DMTs), and cone penetration tests (CPTs) can be performed to supplement existing data. Borings placed next to existing piles can be used for borehole geophysical methods to determine foundation, as discussed in the section on below ground elements.

Surface Geophysical Methods

Wightman et al. (2003) describe five types of surface geophysical methods: potential field methods, seismic methods, electrical methods, electromagnetic methods, and nuclear methods. In general, these surface geophysical methods are employed using equipment at the ground surface to determine the:

- Depth and structure of bedrock
- Extent of bedrock fracturing
- Location of bedrock weak zones
- Physical properties (lithology)
- Location of sand and gravel deposits
- Surface and flow of groundwater

Potential Field Methods

Potential field methods consist of measurements of the gravity or magnetic fields on a site to infer characteristics and stratigraphy of the soil and bedrock. Gravity field methods take measurements of how the gravity field on a site varies spatially. Since the gravitational field is impacted by rock density and depth, this method can be used as a technique for determining how the depth of bedrock varies across a site. Magnetic field methods measure the magnetic field generated by the in-situ bedrock and soil, allowing for the minerology of the bedrock to be estimated. Both methods can be substantially impacted by the presence of existing bridge components, due to their mass and magnetic potential.

Seismic Methods

Seismic methods for subsurface mapping include seismic refraction tomography (SRT), seismic reflection, spectral analysis of surface waves (SASW), common offset surface waves (COSW), sub bottom profiling, and fathometers. These methods generally rely on inputting a seismic excitation into the ground and using geophones located at the ground surface to record the response.

Seismic Refraction Tomography (SRT)

Seismic refraction tomography (SRT) allows for the mapping of bedrock depth along with the shear velocity of the bedrock. Typically, this method is used with an array of geophones spaced in a line away from the input source, usually a dropped heavy weight or powder charge. Shear waves induced by the impact/vibration are transmitted through the overburden soil to the bedrock, then travel in the bedrock as shear waves. These shear waves refract back into the overburden soil and back to the surface. By comparing the time taken to travel multiple distances, the shear velocity and depth of the bedrock can be estimated. This technique can only be used to identify layers that have a higher impedance than the overlying layer. SRT is most viable for shallow bedrock depths.

Seismic Reflection

Seismic reflection methods can use similar excitations as SRT methods and can also use truck mounted vibrators for additional depth penetration. As these input waves reflect off the boundaries (due to a change of impedance) of soil/rock layers, a portion of the energy is reflected up toward the ground surface. By placing a grid of geophone sensors, the depth of reflection can be estimated if the wave speed in the overburden soil is understood. Seismic reflection methods can be used for depths up to 1000m, much greater than with seismic refraction methods. Unlike refraction methods, seismic reflection can be used when underlying soil/rock layers have a lower impedance than the overlying layer, if there is a change in impedance.

Spectral Analysis of Surface Waves (SASW)

Spectral analysis of surface waves (SASW) uses Rayleigh (surface) waves generated from a point source along with a line or array of sensors. Since the wavelength and velocity of Rayleigh waves depends on frequency, the depth of the overburden and the shear velocity of the overburden layer can be estimated using this technology.

Common Offset Surface Waves (COSW)

The common offset surface wave (COSW) methodology uses Rayleigh wave measurements taken from a single excitation source and receiver at time, separated by a specified distance. After the measurement is performed, both are moved while maintaining the same separation and the measurement is repeated. This methodology is best suited or identifying zones of bedrock fracture, as these zones will allow larger, lower frequency Rayleigh waves to pass, delaying the arrival of signal at the receiver.

Sub-bottom Profiling, Acoustic Imaging, and Fathometers

Structural elements and the surrounding ground surface for foundations in rivers, lakes, bays, and other bodies of water cannot always be visually observed, and the exact elevation of the ground surface (Subbottom) may be unknown. Subbottom profiling functions by generating a seismic impact at the water surface and using hydrophones to listen for the acoustic echo off the ground surface. Fathometers work in similar manner, although with this technology typically uses high frequency pings rather than impulses. Acoustic imaging can be used to obtain horizontal views of the foundation in murky water. An acoustic source is directed horizontally at the foundation, and hydrophones are used to perform echo location and imaging of the foundation elements.

Electrical Methods

Electrical investigation methods include self-potential, equipotential, resistivity imaging, and induced polarization methods (Wightman et al. 2003). Self-potential methods measure the potential difference of various areas in a site using a reference electrode. Areas where groundwater is subsiding into rock fissures will generally exhibit lower potential, while areas where groundwater is rising out of rock fissures will experience increased potential. Equipotential methods map the line of equal potential difference from a source input, useful for identifying conductive buried objects. Induced polarization methods characterize soil using the frequency-dependent resistivity properties of various soils, minerals and rocks. These methods have typically been used for environmental investigation, although they can potentially be used to detect certain soil properties. Electrical resistivity imaging has the most potential usability for reuse, as it can map the bedrock surface along a 2-dimensional line and provide some limited information on the properties of the soil and rock.

Electrical Resistivity Imaging (ERI)

Electrical resistivity imaging (ERI) refers to geophysical methods that estimate the resistivity between multiple points on a site. A variety of sensor setups are possible, but commonly ERI involves passing a current between two electrodes at relatively distant points. A second pair of electrodes is then placed in-line between the two electrodes generating current. The interior pair of electrodes is used to measure the potential difference between those two points, which is used to determine the resistivity of the soil and bedrock between the interior points. By performing a series of resistivity measurements, a 2-dimensional map of the bedrock surface can be determined.

Electromagnetic Methods/GPR

The primary electromagnetic method suitable for investigation of reuse potential is ground penetrating radar (GPR). GPR can be used to map bedrock depths, soil moisture content, and subsurface water movement (Wightman et al. 2003). The technology is typically deployed by using a single transmitter in a fixed location and walking the receiving antenna around the site. The transmitted waves reflect off the bedrock and back to the receiving antenna. By estimating the wave speed in soil and knowing the travel time, the distance to bedrock can be estimated. This technology is most suitable for locating bedrock when is it shallow and the overlying soils are not saturated. GPR technology can also be used to estimate the moisture content of the soil.

Borehole Geophysical Methods

Borehole geophysical methods refer to wireline logging techniques that can be employed in completed boreholes or coreholes. The largest advantage to wireline logging is that it can be performed in core holes that are drilled vertically through the foundation and underlying soil. The available technologies are listed in the structural integrity of below ground elements portion, as these technologies can provide a wealth of information about below ground concrete and masonry elements.

CASE STUDIES

Lake Mary Bridge, near Flagstaff, AZ

Overview of Integrity Evaluation

Originally built in 1935, the stone masonry substructure elements of the bridge were extended in 1968 to widen the bridge. Hence, one side of piers and abutment was made of old masonry and other side was of new masonry with concrete pad between the rock and masonry pier elements. The substructure investigation, performed by FHWA R&D, included drilling eight coreholes from the deck through the foundation elements to the underlying bedrock. Locations of these coreholes are shown in figure 31. In this figure, coreholes B_1, B_2, B_7 and B_8 were in the portion of substructure widened in 1968 whereas coreholes B3, B4, B5 and B6 are in the original portion of the substructure. Extensive wireline logging was performed both to assess the foundation and evaluate available technologies. NDT was employed to estimate the integrity and depth of the pier and abutment foundations. The pier and abutment coreholes along with were extended into the limestone bedrock to obtain soil and rock samples.

Structural Evaluation

As part of this investigation, detailed nondestructive and geophysical investigation of bridge components has been performed. Various wireline logging runs have been performed in each corehole. Logging probes used included: Acoustic Televiewer, Optical Televiewer, Full Waveform Sonic, Compensated Density, Electric Log with spontaneous potential (SP)/ single point resistance (SPR), and caliper logs. Figure 31 shows the locations of coreholes. Logging results have shown the presence of voids in the masonry piers and abutments.

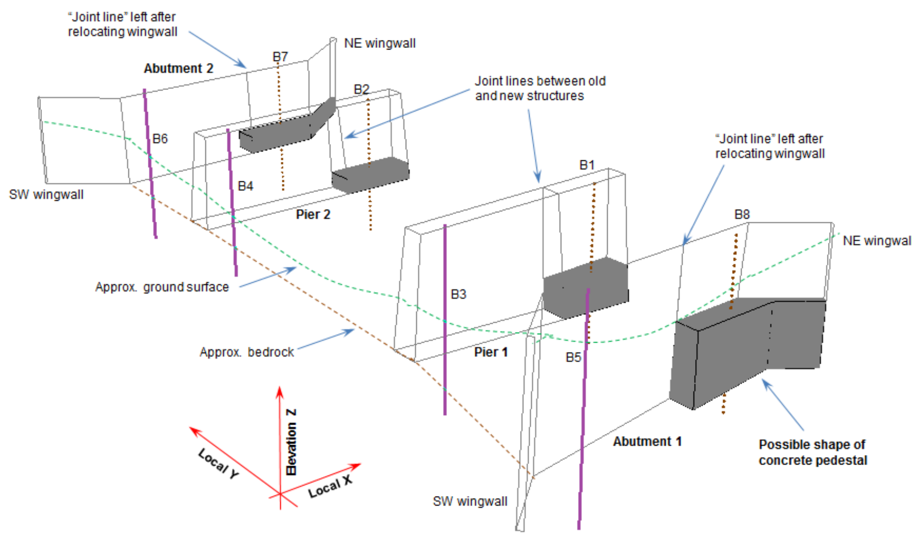


Figure 31. Illustration. Coreholes in abutments and piers in the Lake Mary Bridge

An example logging result, shown in figure 32 shows the composite log of corehole B2 which is in a widened portion of the foundation that includes a concrete pad between the masonry pier and the bedrock. The log shows part of the stone masonry pier, the concrete pad and the underlying bedrock. The log consists of plots/images of the following items left to right: caliper measurements (max acoustic caliper, average acoustic caliper, caliper), optical log, radius image, acoustic televiewer, core pictures, description, and material. Voids are observed in the masonry portion of the optical log between depths from 14 ft. (4.3 m) to 15 ft. (4.6 m). Size of these voids can be observed from the enlargements, which is shaded area in the max acoustic caliper log. These voids are also observed in radius image and acoustic televiewer logs. The condition of concrete in the concrete pad, condition of the bedrock, and interface of the concrete and bedrock can be inferred from the composite log in figure 32. This figure also shows the anomalous areas with reduced velocity. This may be due to voids or deteriorated portions resulting from water seepage from the deck. The result of this investigation was used to plan grouting of voided areas. A detailed finite element analysis showed that the bridge substructure and foundation still had sufficient capacity to carry new load because of widening.

Figure 33 shows composite physical property log for corehole B5 in Abutment 1. This figure shows composite log consisting of (from left to right columns) depth, density log, pressure wave velocity log, sonic log, azimuth diagram for vertical joints, optical log (OBI), picture of removed core samples, material description, and material legend. Presence of an open void is observed near the intersection of the abutment 1 and the bedrock. A large near vertical enlargement/void is seen striking N41.1E. Near this vertical void, the value of density drops drastically because of the presence of deteriorated material in the void. It is noted that the continuous plot of dynamic elastic properties (Poisson's ratio, shear modulus, bulk modulus, Young's modulus and bulk compressibility) can be derived from these logs, as shown for corehole B1 (alongside density, shear and pressure wave velocity in figure 34. Integrity assessment of the foundation element near the corehole region can be made by observing variations in these properties.

As shown, wireline logging provides an in-depth material characterization of the masonry wall foundations around a corehole. For a more volumetric imaging of the foundation elements, seismic travel-time tomography was performed to map the deteriorated (low velocity) zones. Example travel time tomography results are shown for Abutment 1 in figure 35. For this investigation, a swept frequency magneto strictive type source was applied at the exposed long side the pier or abutment wall (red dots in figure 35) and the propagation of seismic energy dominated by the boundary waves was measured by accelerometers. Three-component accelerometers were attached at different locations along the exposed portion of the wall (blue dots in figure 35) and hydrophone string was lowered in the water filled corehole B-8. In general, the incremental values of the boundary wave velocity are proportional to the local elastic properties (shear/rigidity modulus) in the structure, and these values are generally higher in more competent portions and lower in weaker zones.

Travel tomography gave the inverted velocity structure of the foundation wall within the surveyed area—indicated by red lines in figure 36. Volumetric reflector tracing (VRT) technique was next performed to image the foundation elements by using reflection echoes in the seismic record as shown in figure 36. VRT is constructed by migration of reflected wavelets to their reflection points using the velocity model in figure 35. VRT imaging confirms the low velocity zones observed in figure 35 as well as other defective zones outside the (red) tomographic surveyed area. Of interest, is the areal extend of large voids around corehole B-5 above the bedrock (previously observed in figure 47) and the structure of bedrock itself and its dip angle, which is relatively shallow in this case.

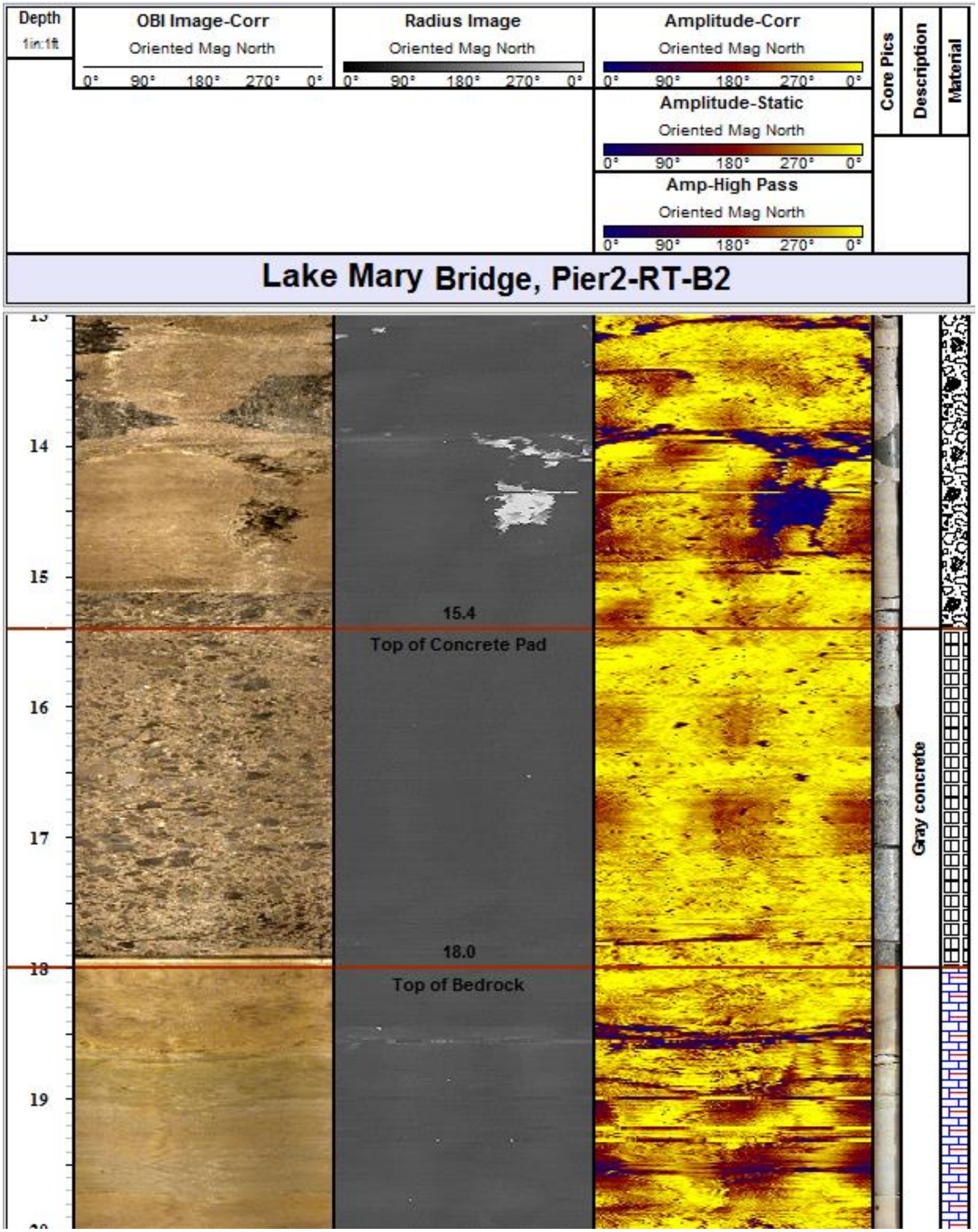


Figure 32. Illustration. Composite log of corehole B2 in Pier 2 of the Lake Mary Bridge

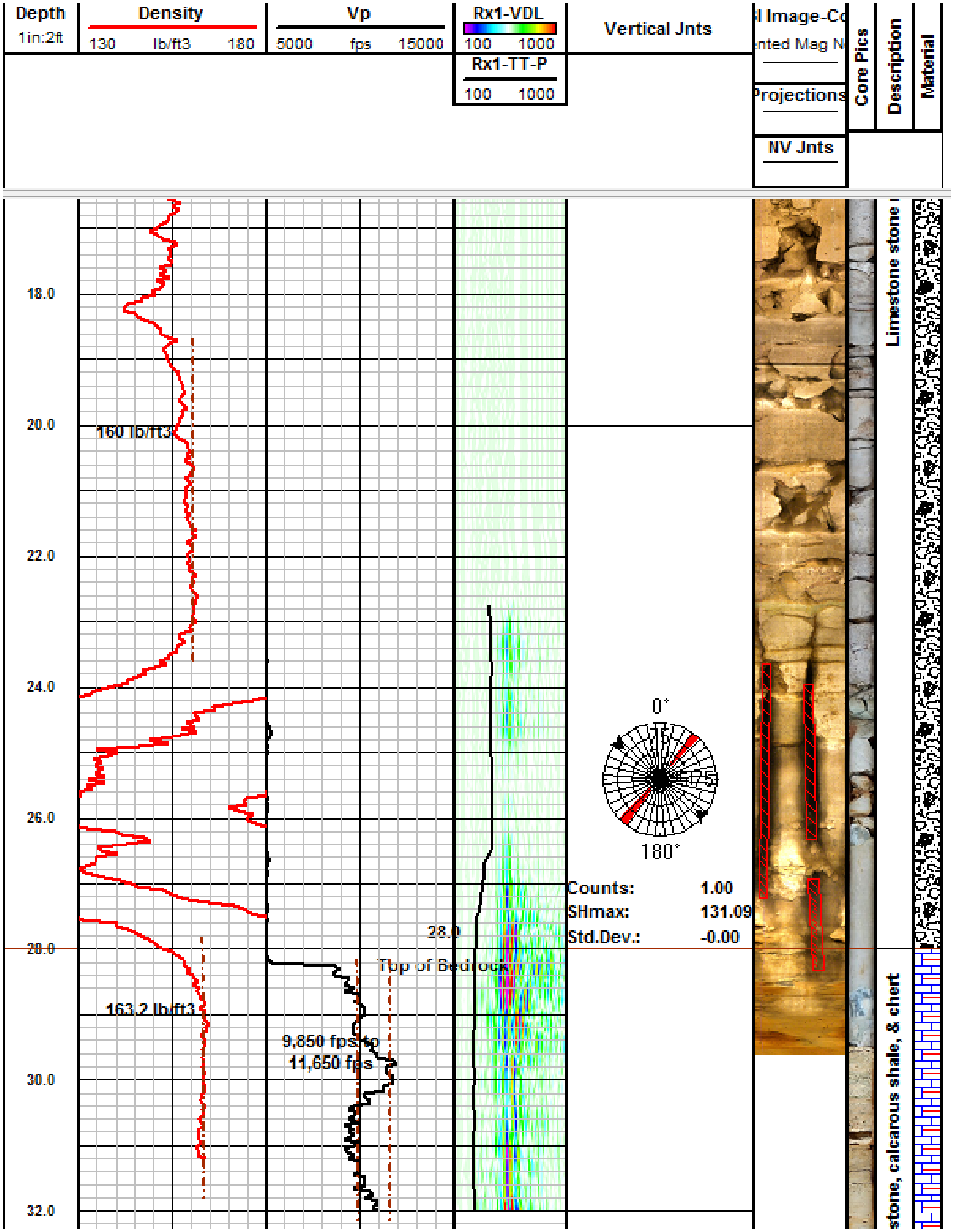


Figure 33. Photo. Physical parameter summary log for corehole B5 in Abutment 1 of the Lake Mary Bridge

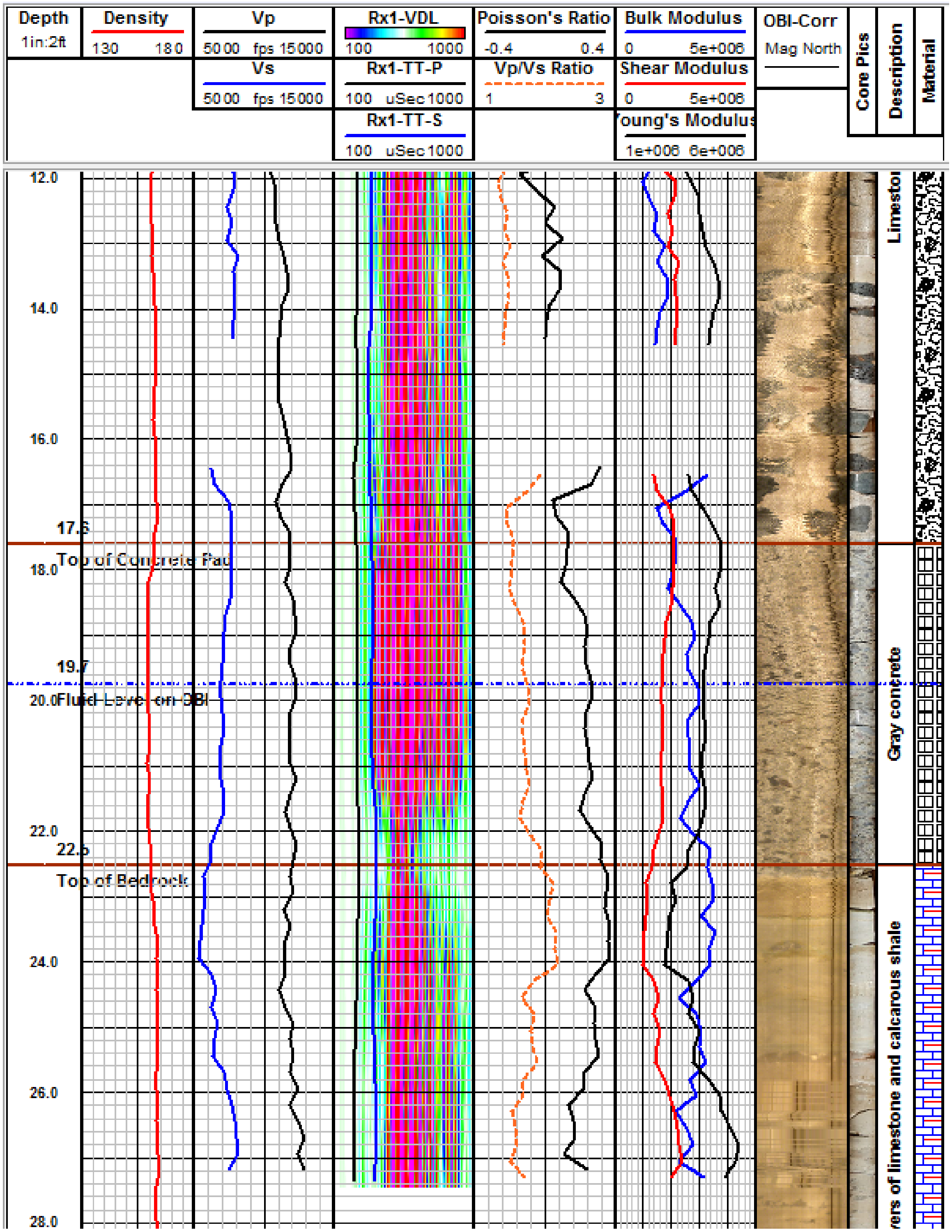


Figure 34. Illustration. Mechanical (elastic) properties log for corehole B1 in Pier1

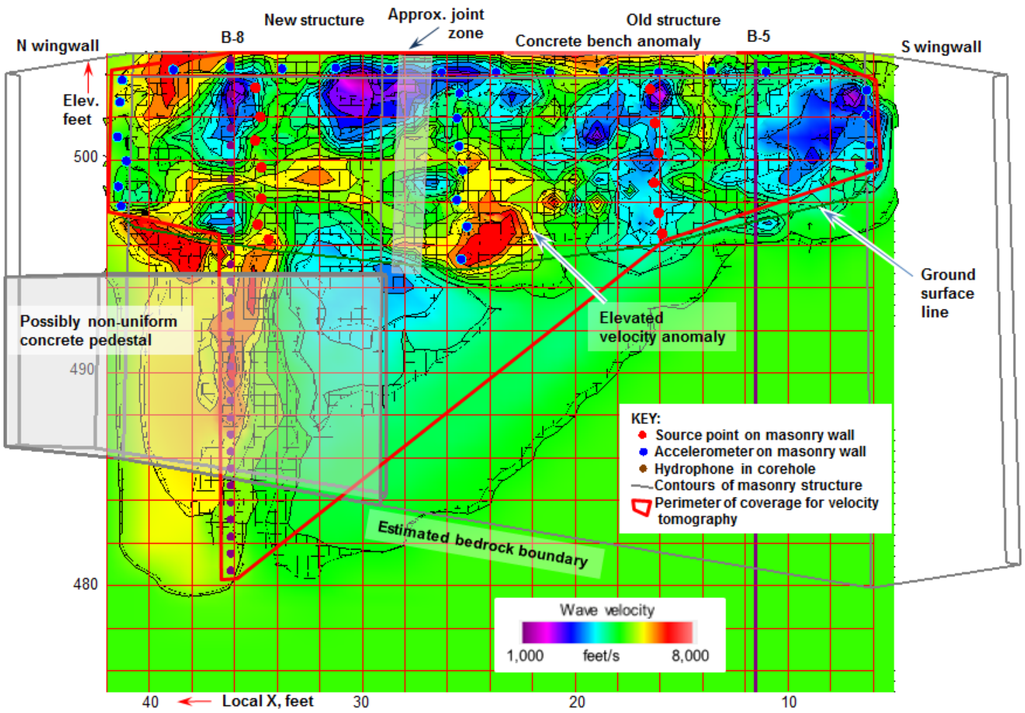


Figure 35. Illustration. Tomogram combined with the volumetric contour image of velocity distribution reconstructed along abutment 1 of the Lake Mary Bridge

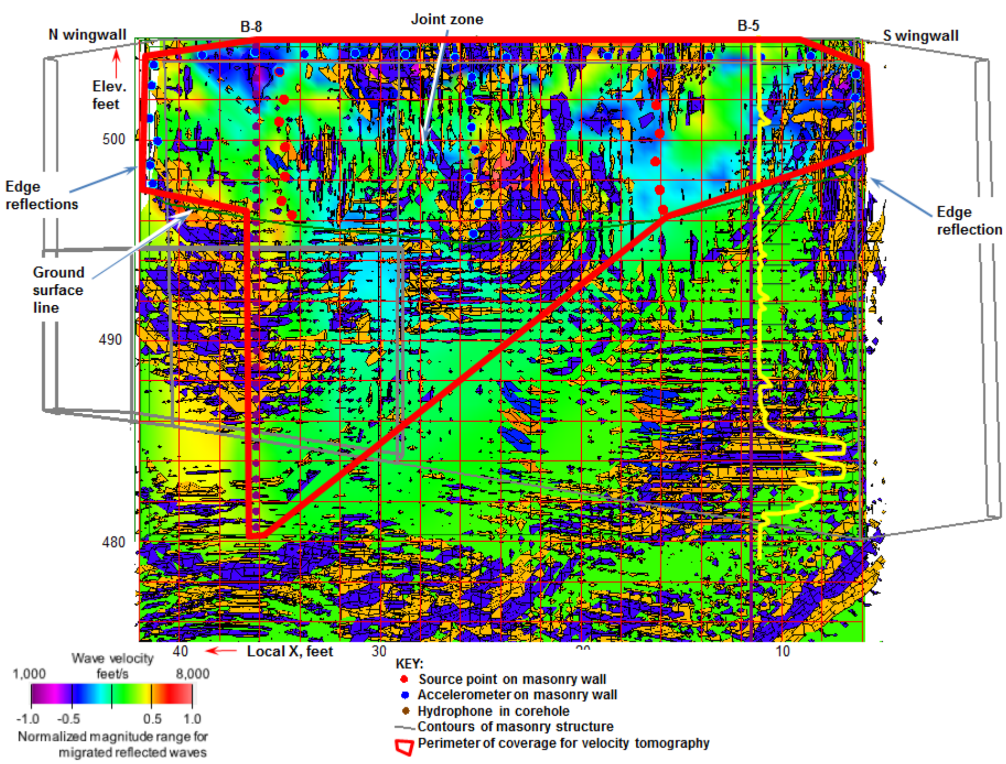


Figure 36. Illustration. VRT reflectogram for abutment 1 superimposed on volumetric velocity model

A summary of the wireline logging and NDT tests performed on the Lake Mary Bridge, what issue was evaluated, and the outcome of that evaluation is provided in table 16.

Table 16. Wireline logging tests performed

	Test	Evaluated Issue	Obtained Result
Logging	Acoustic televiewer	Imaged the inside of the corehole using acoustic device and transducer	Corehole walls were in relatively good condition with little cracking observed
	Optical televiewer	Imaged the inside of corehole using images	
	Full Waveform Acoustic Logs	Determined shear wave velocity by measuring P, S, and Stoneley wave travel time and amplitude	The piers and underlying bedrock were found to be competent. Only minor disturbance was noted at footing/soil interface
	Electric log	Determined electrical resistivity of material	Plots of subsurface resistivity did not identify presence of karst formations near the foundation
	Density log	Log density of material being cored through	Foundation and bedrock did not have zones of very low density
	Caliper Log	Measure shape and diameter of corehole	No major voids or cracking were located
NDT	Seismic Echo	Determine bottom of foundation and located cracking from echo times	Several reflections from cracks/ inclusions were noted, clear measurement of depth was not obtained
	Ultraseismic (US)	Used to determine bottom of bridge pier using time delay of echo and wave travel speed	Could not identify foundation depth
	Ground Penetration Radar (GPR)	Used to identify voids, discontinuities or other features in wall	Multiple weak reflections, no obvious issues identified
	Multichannel Analysis of Surface Waves (MASW)	Determine Shear wave velocity in elements tested	Not effective on masonry piers due to wave refraction along mortar joints

Figure 37 shows a technician performing ultrasonic (US) testing on one of the masonry piers by striking the top of the pier with a hammer while accelerometers are connected to the side of the pier. Figure 38 shows the drilling of a corehole being performed from the bridge deck, while only a single driving lane in closed.

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Figure 37. Photo. Performing ultraseismic testing on a pier

Source: FHWA

Figure 38. Photo. Coring through pier from bridge deck

Geotechnical Evaluation

A geotechnical test program consisting of geophysical logging, rock coring, and testing of removed samples was conducted. The geophysical methods employed consisted of Electrical

Resistivity Imaging (ERI), Seismic Refraction Tomography (SRT), a summary of which is given in table 17.

Table 17. Geophysical testing performed

Technology	Purpose	Results
Electrical Resistivity Imaging (ERI)	Identifies resistivity of soil and bedrock by placing probes into borehole and onto ground surface	Rock showed a resistivity of over 50 Ohm-m, while soil had lower resistivity of about 5 Ohm-m. No major holes in rock were found near foundation.
Surface Analysis of Seismic Waves (SASW)	Measures shear and pressure wave velocity to determine density and modulus of soil/rock	No loose zones of rock found near coreholes.
Seismic Refraction Tomography (SRT)	Look for bedrock surface using P-Wave and S-Wave refraction	Interpreted bedrock line using results from SRT

Milton Madison Bridge, between Madison, IN and Milton, KY

Overview of Integrity Evaluation

The initial condition assessment found horizontal cracking, vertical cracking, and lift lines in the piers being investigated for reuse. Varying amounts of spalling were observed, and zones of delaminated cover concrete were identified through hammer sounding. During this evaluation, six 4-inch (100 mm) diameter cores were taken for compression testing from the face of four piers, including three of the four piers eventually reused. An additional eight cores from the faces of the piers were taken for petrographic analysis. Petrographic analysis was also performed on seven cores taken vertically from the pier. The cores were drilled from the bridge deck ledges in the substructure. Logging of the vertically drilled cores was not performed. GPR and IR were performed to determine the location of reinforcement, cracking and delaminations. Pictures taken of the conditions at time of testing are provided in figure 39.

A. cracking along the face of the pier B. delamination of the cover concrete

Figure 39. Photo. In depth inspection photos of pier 7 of Milton Madison Bridge

The initial evaluation also identified areas of minor paste erosion near the water line, possibly due to the presence of lower quality or weaker concrete. Lift lines were observed that indicated cold joints were present from the initial construction. Deterioration of the concrete above and below some of the cold joints had led to exposed steel rebar that became a focal point for corrosion, as shown in figure 40.

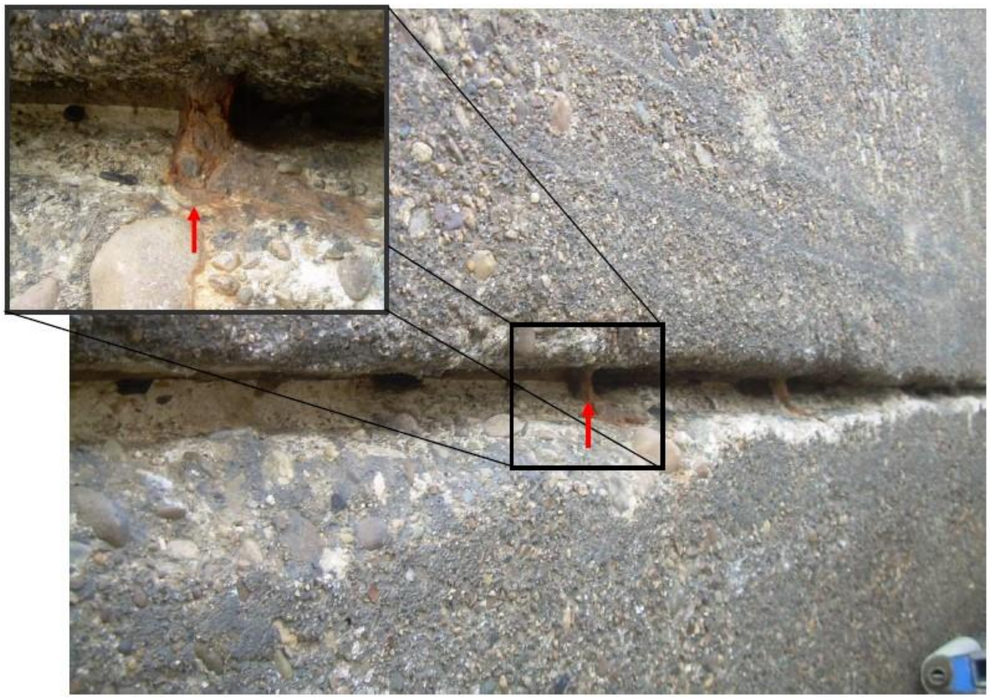


Figure 40. Photo. Lift line with concrete deterioration leading to exposed rebar

Considering the observed concrete deterioration, a program involving petrography and compressive strength evaluation was proposed for integrity testing, along with chloride profile analysis, as discussed in Chapter 5. To perform the cores required for this testing, it was necessary to locate and confirm the layout of the existing reinforcement to avoid accidentally drilling through it. A summary of the issues identified during the initial evaluation, and how they were identified or the background reason for the concern is provided in table 18.

Structural Evaluation

The primary goals of the testing program were to evaluate the concerns noted in table 14, notably the quality of the interior concrete and the quality of the concrete cover. Testing of the interior concrete focused on the drilling of coreholes through the center of the concrete piers proposed for reuse. The installation of coreholes allowed direct sampling of the concrete from the entire depth of the pier, including the caisson and underlying rock. After completion of the corehole, the interior of the concrete was further inspected using a camera for visual downhole observations. Single-hole Sonic Logging (SSL) (a simplified form of full waveform sonic used in geophysical logging) was performed using a hydrophone lowered into the completed core hole to estimate mechanical properties and confirm the intactness of the pier. Petrographic analysis was performed on 7 samples taken from the vertical coreholes.

Table 18. Items of concern, and how the concern was identified

Item of Concern	Initial identification/ reason for concern
Pier Concrete Cracking	Noted during in-depth inspection and corrosion inspection. At least some of cracking appears to be from early age
Spalling, Delamination, rebar corrosion of Pier Concrete	Spalling noted through visual survey. In-depth inspection identified delaminated areas with hammer sounding. Delamination occurred over much of some piers and limited sections of other piers
Lift lines (cold joints)	Evidence of cold joints at interface of multiple concrete lifts. Deterioration noted at cold joint near water surface
Paste Erosion	Visual indications of erosion of concrete paste were visible from outside the pier at the water line.
Rebar Layout, cover depth	On design plans, no confirmation of exact rebar layout or depth
Concrete Quality	Unknown current strength and variability of concrete. Unable to assess quality of interior concrete through visual/physical methods alone. Exterior concrete appeared to be of variable quality during in-depth inspection and corrosion evaluation.

Testing of the exterior concrete was performed with Impulse Response (IR) testing and a Ground Penetrating Radar (GPR) survey. The IR testing allowed for complete mapping of areas with delaminations or substantially deteriorated concrete. The GPR survey allowed for mapping of the underlying rebar and the cover depth. The mapping of rebar was essential to avoiding damaging the reinforcement with cores taken from the faces of the piers. Coring of the pier faces was performed at 13 separate locations. During this evaluation, six 4-inch (100 mm) diameter cores were taken for compression testing, including from three of the four piers eventually reused. An additional eight cores from the faces of the piers were used for petrographic analysis. Figure 41 and Figure 42 illustrates impulse response results and core samples obtained during investigation, respectively figure 43(a) shows the drilling procedure through the deck and figure 43(b) shows drilling from a barge in the Ohio River.

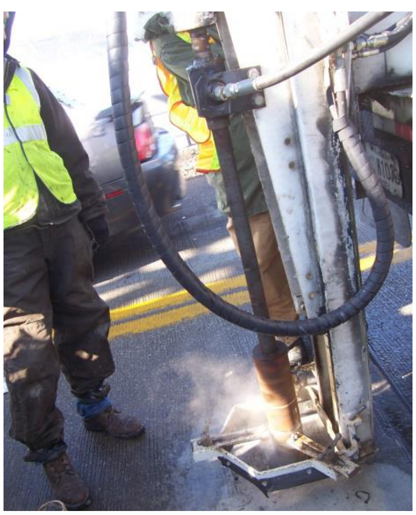
Concrete cores were extracted from both the faces of the concrete piers and from core holes drilled vertically through the deck, pier, and underlying rock. In all, 54 compression tests were performed on 2 in (50 mm) diameter cores removed from the vertical coreholes, and six compression tests were performed on 4 in (100 mm) diameter cores removed from the pier faces. The cores taken from the vertical coreholes had an average unconfined compressive strength of 9,250 psi (64.8 MPa), with a standard deviation of 1,780 psi (12.3 MPa). The test breaks for all the cores were generally above 6,000 psi (41.4 MPa), with 1 test breaking at 4,810 psi (33.2 MPa). The six 4 in (100 mm) cores had unconfined compressive strengths ranging from 7,000 psi (48.3 MPa) to 13,750 psi (94.8 MPa), with an average of 10,297 psi (71 MPa). Static modulus of elasticity testing was performed on two samples taken from the pier faces, which were found to have moduli of elasticity of 864,000 ksf (41.4 GPa) and 907,200 ksf (43.4 GPa). These values exceeded the material properties used in design, but ultimately the design modulus of elasticity was taken from the unconfined compressive strength. Table 19 summarizes the tests performed, issues evaluated, and extent of testing for this bridge.

Figure 41. Photo. Impulse response test results

A. Sample cross section

B. Sample elevation

Figure 42. Photo. Cored samples



A. Through the deck

B. On the Ohio River

Figure 43. Photo. Drilling in Milton Madison Bridge

Table 19. Tests performed, issues evaluated, and extent of testing

Test Performed	Issues Evaluated with Testing	Extent of Testing	Outcome
GPR	Cover depth, Rebar layout	135 scans performed along 3 reused piers and 1 pier not reused	Drilling locations were chosen to avoid rebar. GPR survey confirmed very low reinforcement ratio.
Impulse Response	Extent of delamination, check for indications of honeycombing and subsurface voids	Performed along 3 reused piers and 1 pier not reused	Area of higher reflectivity corresponded to delaminated areas of concrete.
Down-Hole Camera in Core Hole	Integrity of interior concrete	1 core on each of 4 reused piers	No significant voids observed, ineffective below water line
SSL on Core drilled through pier	Determine if voids or defects are present in pier	1 core on each of 4 reused piers	Anomalies reported just above caisson/soil interface, few other minor anomalies
Petrography	Mix properties; damage from freeze/thaw, erosion, ASR, carbonation penetration	Performed on 15 core samples, some extracted from exterior face, some from vertical coring through center of pier	Mix properties verified, minimal freeze/thaw or ASR noted, little carbonation penetration
Compression Tests	Compressive Strength	58 compression tests, 54 on 2" cores, 4 on 4"	Compressive strength taken as 1.5 standard deviations below average of 2" tests
Modulus Tests	Modulus of elasticity for concrete	Performed on 2 4" dia. cores, also taken from concrete strength found from compression testing	Modulus used was from strength-based calculations

Geotechnical Evaluation

Borings were conducted in two phases: an initial round of 4 land borings and 4 borings through pier elements; and an additional round of land borings, river borings and CPT soundings. The second round of testing was devised and carried out after it had been decided that Piers 6 – 9 would be reused. Beyond the concrete sampling, the borings from the initial explorations were used for rock unconfined compressive strength testing, soil classification, and rock mass rating. Triaxial compression testing and consolidation testing were performed on soil samples taken from the second round of borings. Nine CPT soundings were performed between the sides of the river until refusal. The CPT soundings directly measured cone tip resistance, side resistance, and excess pore pressure, from which the undrained shear strength, the N60, the density index, the angle of internal friction, and the constrained modulus were interpreted. All the CPT soundings were terminated above the bedrock, in a dense layer of soil likely containing cobbles and boulders.

The original superstructure had not seen substantial amounts of foundation movement or settlement during its lifespan. While located in a major waterway, existing scour was not noted on any of the reports, and the pier remains well embedded in up to a 60-ft (18 m) deep sand and gravel layer. Large boulders and cobbles were noted in this layer during caisson construction, and evidence of cobbles or boulders was found in the additional borings performed. Ten borings from the original investigation were found, 9 of which included rock cores. An additional test program was conducted that included 22 borings on land, 12 borings in the river and 4 coreholes through the pier into the underlying rock. CPT soundings were performed at 9 locations to assess the penetration resistance and the depth to bedrock.

Conclusions

In total, four of the five piers investigated were considered suitable for reuse. Minor ASR was found through petrography, but no associated damage was observed. No extensive freeze damage was present, although the concrete mixture was considered vulnerable to future freeze thaw action due to the lack of air entrainment noted during petrography. The extent of cracking, spalling, and delamination of the cover concrete raise durability concerns discussed further in Chapter 5. Cold joints along old lift lines did not show good adhesion when cores spanned this boundary, though the existing cold joints were expected to be primarily horizontal. The extent of rebars found was very low by modern standards and additional reinforcement was needed.

The coring showed few voids in the internal pier concrete and the compressive strength of the concrete was deemed to be 6,850 psi (47.2 MPa). The strength was based on 54 compression tests of concrete cored from the center of the pier, with an average strength of 9,250 psi (63.8 MPa), and a standard deviation of 1,780 psi (12.3 MPa). The design compressive strength of 6,250 psi (43.1 MPa) was 1.5 standard deviations below the mean. The evaluation of the cover concrete showed compressive strengths higher than the design value, but with obvious signs of decay and construction issues. Cold joints showed deterioration around the outside edges, even exposing rebar.

The scour investigation identified a potential of 2.0 ft. (0.6 m) of contraction scour and up to 39.7 ft. (12.1 m) of local scour around the piers for a 100-year flood. A 500-year flood produced contraction and local scour depths of 3.0 f.t (1 m) and 41.1 ft. (12.5 m), respectively. The rock underlying the caissons was found to have a Rock Mass Rating of 58. A nominal bearing resistance of 75 ksf (3.6 MPa), obtained by using a Hoek-Brown model, was used as the rock bearing capacity.

North Torrey Pines Bridge, Del Mar, CA

Overview of Integrity Evaluation

At the North Torrey Pines Bridge, the exposed substructure elements had shown significant signs of deterioration with extensive spalling and delaminations spanning back a decade (Johnson et al. 2015a, 2015b). The current pier had obvious deficiencies and has shown continuously worse deterioration. The north faces of the substructure elements (the ones directly exposed to sea air) were found to have 3.1 percent of their surface damaged by delaminations, while the remaining faces ranged from 0.43 percent to 1.5 percent damage. Approximately 35 percent of the concrete surface had suffered some form of concrete cover damage.

Structural Evaluation

New as-built plans were drawn up for the visible portions of the foundation from in-situ geometry measurements. GPR was used to estimate cover depths on at least 16 locations per column on a total of eight columns spread over six bents. The cover depth was found to be an average of 2.85 in (72 mm) for the horizontal reinforcement and 3.56 in (90 mm) for the vertical reinforcement.

The minimum cover depths were 1.04 in (26 mm) and 1.65 in (42 mm) for the horizontal and vertical reinforcement, respectively. The horizontal (shear) reinforcement was found to provide unacceptable confinement by modern earthquake resistance standards.

Compressive strength and modulus testing was performed, and the test results found an average compressive strength of 4200 psi (29 GPa). The static modulus of elasticity was found to be 3700 ksi (25.5 GPa). The original design plans had specified the use of 4000 psi (27.6 GPa) concrete, so the test data was consistent with expectation and allowed for a slight increase in strength. The precast piling was assumed to have a compressive strength of 5000 psi (34.4 GPa) and a modulus of elasticity of 4000 ksi (27.6 GPa), as per design plans.

A 0.5 in (13 mm) wide bar was removed the structure and load tested to determine steel properties. The yield stress of the bar was found to be 46 ksi (317 MPa), higher than the specified yield strength of 40 ksi (276 MPa). The ultimate tensile strength was found to be 76 ksi (483 MPa), rather than the specified 70 ksi (483 MPa).

Geotechnical Evaluation

A boring program was implemented to identify potentially liquefiable soils. 16 borings, 5 CPT soundings, parallel seismic logging, SASW, and seismic CPT soundings were performed. The SPT and CPT data was used to estimate liquefaction potential given the site's potential ground motions. Liquefiable areas were near various pile bents and nearby embankments. The liquefaction potential was considered a hazard for the foundation performance and the nearby embankment stability.

Georgia Street Bridge, San Diego, CA

Overview of Integrity Evaluation

Significant material testing and structural evaluation of the Georgia Street Bridge was performed in late 2011 and early 2012. The bridge had a documented history of evaluation prior to this, including an extensive evaluation of the retaining walls outside of the bridge abutment in 1995. Petrographic analysis during the 1995 study had found low cement content, a high w/c ratio of the initial concrete, and high permeability. It was noted at that time that the northern wall was in too poor of shape to allow for the extraction of intact cores. Rebar extraction and testing had found that some reinforcement had lost up to 30 percent of the original bar size due to corrosion. A 2009 study conducted unconfined compressive strength testing and again noted that in several areas the concrete was of too poor quality for intact cores to be extracted.

The previous investigations had found evidence of poor initial quality, rebar corrosion, excessive permeability, and general deterioration of the concrete surface. Multiple layers of shotcrete existed in areas which had been previously repaired. A summary of the concerns is provided in table 20.

Table 20. Items of concern and how concern was identified or underlying reason

Item of Concern	Initial Identification/Reason for Concern
Concrete Quality	From earlier studies: North wall concrete not stable enough for coring, south wall concrete had low w/c ratio and cement content found from petrographic analysis.
Concrete Strength	Strength testing available from select areas of retaining walls outside of the abutments.
Rebar Strength	Unconfirmed yield strength of rebar, corrosion of reinforcement (up to 30% loss) had been noted previously

Structural Evaluation

Table 21 presents the testing performed at the Georgia Street Bridge, what issues were evaluated, the extent the testing was performed, and the outcome of the testing.

Table 21. Testing performed, and issues evaluated

Test Performed	Issues Evaluated with Testing	Extent of Testing	Outcome
GPR	Rebar layout	Performed in proposed core locations to avoid	Coring locations were chosen to avoid rebar. GPR survey confirmed very low reinforcement ratio.
Rebound Hammer	Gauge consistency and soundness of surficial concrete	Performed in grid along abutment face, arch ribs, and cross beams	Area of higher reflectivity corresponded to delaminated areas of concrete.
Compressive Strength Testing	Determine strength from additional locations	Thirteen 4 in (10 cm) dia. cores tested, 8 in the arches and cross beams, 2 in both the north and south abutment	Abutment cores had compressive strengths of 2330 psi (16 GPa), 2890 psi (20 GPa), 2940 psi (20.2 GPa), and 3810 psi (26 GPa).
Petrography	Analyze concrete composition, extent of carbonation, presence of ASR	Six 6 in (15 cm) dia. cores extracted, 1 from each abutment. Tested for ASR, carbonation, and chloride penetration	No ASR detected in abutment, minor ASR detected in 1 superstructure core.
Rebar Testing	Test yield strength and ultimate strengths of rebar, elongation at failure	One 24 in (61 cm) long sample removed from each abutment	Both bars had yield strength just over 36 ksi (248 MPa). Ultimate strength was 53 ksi (365 MPa) and 57 ksi (393 MPa).

Geotechnical Evaluation

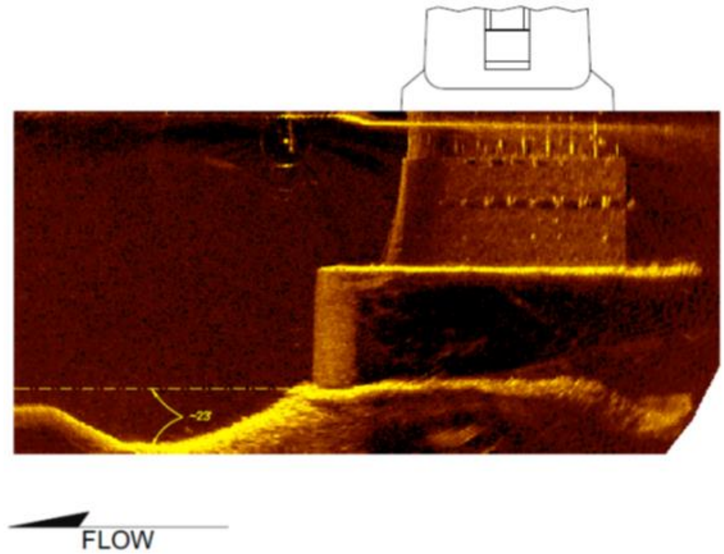
The geotechnical evaluation consisted of borings, test pits, and soil testing. Historical photos, design documents, and the geotechnical evaluation were used to determine that the abutment walls had been cast against a weak sandstone on the lower portions and retained a compacted fill on the upper portions. The friction angles and cohesion of both the fill and the sandstone were determined in the geotechnical evaluation.

Huey P. Long Bridge, Jefferson Parish LA

Overview of Integrity Evaluation

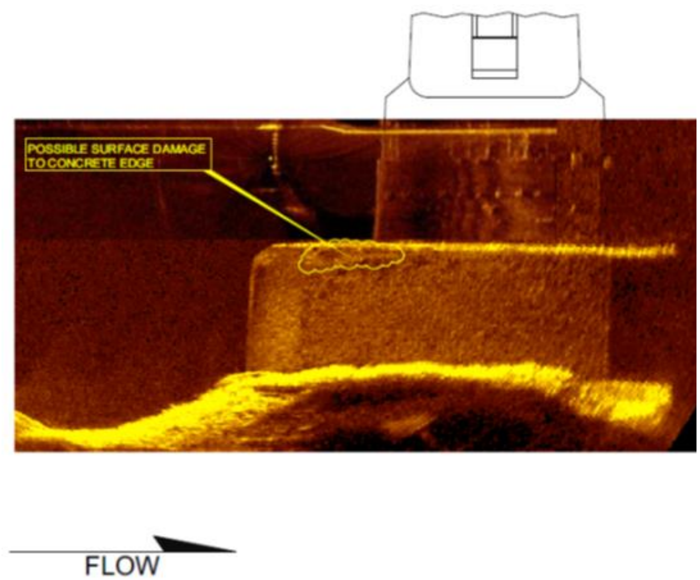
A diving inspection was performed to evaluate the condition of the pier below the water line. Due to the fast-moving currents and low visibility, the visual component of the diving inspection was of little use. Acoustic sonar imaging was performed to visualize the condition of the pier as well as the soil surrounding the pier. Examples of the images obtained through acoustic imaging are provided in figure 44.

A. North face with regular image



B. North face with sonar image

C. South face with regular image



D. South face with sonar image

Figure 44. Photo. Acoustic images of Huey P. Long Bridge pier

Structural Evaluation

The concrete strength was determined without the use of additional testing. The concrete test break values from the initial construction were used, in conjunction with expected increases in capacity, as discussed in Chapter 6. Other than strength concerns, there were no obvious signs of deterioration, including in the underwater portions of piers that were investigated with acoustic imaging.

Geotechnical Evaluation

A geotechnical investigation was performed to determine the properties of the soil underlying the foundation. A total of ten borings were conducted directly adjacent to five piers, generally finding a hard clay overlying a dense, fine sand layer. Undisturbed samples of the underlying soil were taken for testing and analysis. Since the borings were performed directly next to the existing piers, they could locate the areas where soil had been disturbed by the cutting head used during caisson installation. It was determined that 4 of the bridge piers were installed well into the competent sand layer, while the fifth pier was installed to the estimated depth of the sand layer. The pier settlements were monitored from 1940 through 1990, showing no long-term settlement outside of the initial elastic settlement. All bridge piers within the river experienced scour, with one experiencing up to 26 ft of scour. This scour had not undermined the foundation, and the missing soil was not included in the overturning resistance.

The approach spans were founded on timber piles, some of which were reused, and some that were abandoned. Complete pile driving records were available so there was not any uncertainty about the installed depth and length. The piles were also well below the long-term water level, meaning the timber piles were not expected to be exposed to dry conditions. During construction, when one of the pile bents was removed, timber piling that was no longer needed was exposed and observed. The piling was observed to be in sound condition with no external decay, splitting, or crushing noted. The removed piles were sounded to confirm there was no internal decay present. This investigation concluded that the remaining timber piling on the site would continue to be in suitable condition assuming they remained below the water table.

NJ Route 72 Bay Bridges, Ocean County, NJ

Overview of Integrity Evaluation

The piles underneath the main-span bridge were permanently submerged and buried underground and under a concrete pile cap. Additionally, the reduction of lanes carried by this bridge from four to two lowered the live loading on these piles. The critical foundation elements to consider were the timber piles underneath the trestle bridges that were subjected to alternating wet/dry conditions.

The dimensions (diameter and embedment depth) of every existing pile underneath the trestle bridges were determined through measurements and nondestructive testing (NDT). Sampling of the piles allowed for species determination and the allowable stresses in the piles were determined in accordance with ASTM Standard D2899 (ASTM 2012). The amount of section loss in the trestle piles was measured for every pile through coring and sampling. A scour hole approximately 8 ft. (2.4 m) in depth had been located at one of the bents during routine underwater inspections.

Structural Evaluation

Pile diameter was determined by measuring the circumference at each exposed pile. NDT, primarily sonic echo (SE), was used to confirm the depth of the driven piles with as-built plans and design drawings. The sonic echo testing confirmed that the tested piles are longer than 65 ft (19.8 m), which is the approximate depth of the anticipated bearing stratum.

Geotechnical Evaluation

Soil data was available from previous borings and inspections. This data was used to determine the depth to bearing strata, the grain size distribution, and other soil properties. Measurements were taken around the piles during previous inspections to estimate the approximate extent of existing scour holes, and the unbraced length of the existing piles.

Haynesville Bridge, Haynesville, ME

Overview of Integrity Evaluation

The primary concerns related to the reuse of the Haynesville Bridge foundation were the length and integrity of the existing treated timber piles. The original design drawings specified treated timber piles with a design pile length of 35 ft (10.7 m) for the abutments and 25 to 30 ft (7.6 to 9.1 m) for the interior piers. The water level was considered to fluctuate with seasonal variations but was approximately 8 to 9 ft (2.4 to 2.7 m) below the bottom of the concrete abutments, occasionally exposing the top of the timber piles to non-submerged conditions.

Structural Evaluation

The condition of the timber piles was analyzed by exposing them through full excavation of the abutments. The upper most 20 in (51 cm) of the timber was sawed off, with the testers noting the presence of creosote during cutting. The exposed top of the remaining pile was then tested with a Pile Integrity Test (PIT) that consisted of striking the pile head with a hammer and waiting for a response. A pile integrity test was performed on a single pile from each abutment, finding that the piles had been driven to 26 ft (7.9 m) and 32 ft. (9.7 m). Figure 45 shows the excavated abutment with a pile being cut. Static load testing was then conducted on the piles exposed under the two abutments, as discussed in Chapter 6.

Figure 45. Photo. Excavation and removal of portion of timber piling at Haynesville Bridge

Geotechnical Evaluation

Four test borings were conducted at the site to bedrock, with cores of the bedrock extracted. Rock Quality Designation (RQD) was performed on the removed cores and SPT blow counts of the soil were recorded. The soil properties found were used to estimate the nominal capacity of the driven piles at site.

U.S. Route 1 Viaduct, Bath, ME

Structural Evaluation

Overall, eleven piers and one abutment were supported by steel H-piles. Of these, deep test pits were excavated at four of the abutments to directly observe the steel piles (figure 46). In general, no corrosion was observed, except for a few spots with minor chipping or pitting. It was determined that in general, the water table remained above the bottom of pier cap for most piers, with some piers having the water level just reach the bottom of the pier cap. It was determined that corrosion was unlikely to have occurred on the steel piles, however a 1/16-inch (1.6 mm) reduction to the cross-sectional area of the pile was recommended to account for their extended service life.

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Figure 46. Photo. An exposed steel pile in a test pit

Geotechnical Evaluation

To confirm the depths of the piles, additional borings were performed to supplement the previously obtained data on the subsurface conditions. Parallel seismic (PS) testing was performed in the completed boreholes at 6 of the 12 foundations where steel piles were being considered for reuse. Test pits were excavated at these six foundations so that the structure could be impacted. Reasonable confidence in the depth was found from three of the tests conducted. Test on remaining three foundations were inconclusive because of noisy results, two very different possible depths, or not producing meaningful results.

Mississagi River Bridge, Ontario, Canada

Overview of Integrity Evaluation

The integrity evaluation consisted of an underwater inspection of visible components, as well as a coring program through each of the four riverine piers. The inspection program evaluated the condition of the sheet pile cofferdam and the amount of scour observable outside of the cofferdams and footings. Ultrasonic inspection was also employed to evaluate the condition of the sheet piling. The coring was performed from the bridge deck, alongside the piers, and into the top of the footing through a ledge on top of the footing. The coreholes were advanced through the footing concrete and into the underlying soil. The documentation of the original piers had not included an installed depth, so the coreholes were used to definitively determine the foundation depth.

Structural Evaluation

The sheet piling was evaluated through visual and ultrasonic inspection. These inspections identified areas of corrosion in the sheet piling, especially underwater near the mudline. The concrete exposed in underwater portions where the sheet piling had corroded was in observably poor condition. Standard investigations were performed on the piers (above the footings) that indicated the pier concrete for all piers was in good condition with at least the original design strength.

The primary data recovered about the footing concrete through coring was the RQD. Piers 1, 2, and three generally had a recovery of approximately 15 percent, while Pier 4 had an RQD of 100 percent. The coreholes also verified the depth of the footings by determining the depth of the concrete/soil interface.

Geotechnical Evaluation

The underwater inspection observed various areas of scour in the stream bed outside of the cofferdams. The observed scour had not undermined the sheet pile cofferdams but was lower than the bottom of footing elevations noted for several of the piers. This, along with the corrosion noted in the steel piling prompted concerns that further corrosion of the sheet piling could lead to a loss of confinement in the soil supporting the piers and undermine the foundations.

The coring that was performed through the pier footings and into the underlying soil found that Piers 2 and 3 were supported on a loose to dense sand and gravel. Pier 1 was founded on a silt and sand layer that overlaid the sand and gravel stratum. Underneath this sand and gravel stratum, a seam of approximately 5m thick material that ranged from clayey silt to silty clay was identified. Testing performed on soil samples from the underlying layers, including SPTs and sample recovery indicated the foundations did not have adequate capacity.

Henley Street Bridge, Knoxville, TN

The integrity evaluation at the Henley Street Bridge consisted of identification of structural issues (cracking of spandrels, corrosion of steel, spalling, and section loss of concrete), determination of cover depth for the arch ribs and spandrels, and a geotechnical exploration including borings and soil sampling/testing. A total of 20 borings was performed, including retention of three rock samples and three soil samples.

Jackson Road Bridge, Lancaster, MA

The integrity evaluation program consisted of four test pits (figure 47), one in each corner of the two abutments. The pile integrity of the timber piles was evaluated through visual inspection, coring, resistograph testing, and pulse echo testing. Visual inspection of the piles exposed in each test indicated that the piles were in good condition. Three 1/2-in (13-mm) core samples taken from a test pit pile at 8.5 in (21.6 cm), 18 in (46 cm), and 28 in (71 cm) below the pile cap. The core samples identified the preservative penetration depth (4.75 in (12 cm) for all cores), the depth of the pith, and a visual observation that no decay was present. Microscopic evaluation of the cores indicated that there was no fungal growth and identified the species of wood present. Resistograph testing was performed in 36 locations that supported the findings observed in the core-tested pile and ensured that the remaining piles did not have excessively weak zones. Pile integrity testing (pulse-echo) was performed to confirm that the pile length below grade and into the pile caps matched the documented lengths in the pile driving records. No major reflections that could indicate damage or voids were found from the pulse-echo data. Ground water was not encountered during any of the test pits, indicating that the water table was beneath the project site. Due to the observed good condition of the timber piles, it was assumed they would remain serviceable for the life of the replacement project, and a remaining life analysis was not undertaken.

A. Typical test pit setup with trench box

B. Timber pile at test pit 2

Figure 47. Photo. Integrity assessment of timber piles in Jackson Road over MA Route 2 Bridge

Crowchild Trail Bridge, Alberta, Canada

The existing piers have displayed a history of “good” performance, used to justify the continued usage of the piers. As-built documentation for the existing piers is available and gives structural details on the pier geometry and material properties. The piers have not undergone any obvious deterioration that warrants additional material testing and analysis. The initial investigation for reuse of these piers therefore has focused determining the geotechnical conditions and capacity at the pier sites. Primarily, borings have been used to sample both the soil and rock, as well as verify the stratification of these layers. Testing has been performed on rock samples to quantify the strength and compressibility of the bedrock (figure 48). In addition to the boring and sampling program, a test pit was conducted on the riverine pier exposed during low-flow (Pier 1.) The test pit identified that the footing was in good condition, the pier had not suffered from previous scour, and the geometry match the as-built drawings.

Figure 48. Photo. Drilling borehole for obtaining soil samples

CHAPTER 5. DURABILITY AND REMAINING SERVICE LIFE

INTRODUCTION

This chapter covers the evaluation of the durability of an existing bridge foundation being considered for reuse. Two of the questions most at the forefront of foundation reuse are: how much remaining service life does the foundation have, and will the advanced age of the reused components increase life cycle costs. Modern code provisions for new structures provide specifications on material properties and details to ensure adequate service life and limit life cycle costs. Some issues, like chloride ingress or carbonation in concrete may have reduced the remaining life without yet causing noticeable damage. Corrosion of steel piling may have reduced the amount of cross-section available and therefore the capacity. Previously identified issues that have been repaired (i.e. spalling, delamination, etc.) may still impact the durability if the underlying problem still exists or if the repair itself has a limited service life. In many cases, strengthening is employed to aid with durability and capacity issues simultaneously, by repairing existing damage or planning for future deterioration that lowers the capacity of the damaged element.

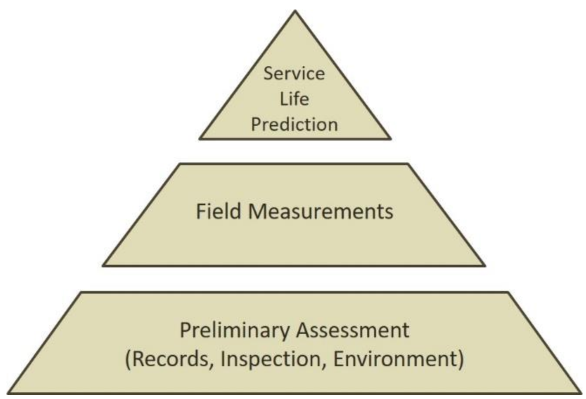

Figure 49. Illustration. Stages of durability assessment

The durability assessment for bridge substructures is roughly grouped into three parts: the preliminary evaluation, field measurements and testing, and service life prediction (see figure 49). The purpose of performing the condition assessment in three parts is to minimize expenditures related to testing and evaluation. The goal is to identify primary durability concerns, perform testing relevant to those concerns, and then assess the service life and life cycle costs using data acquired during testing, when needed.

ISSUES CONSIDERED DURING DURABILITY ASSESSMENT

Concrete Elements

The most common concrete elements found in bridge foundations are wall piers, columns, driven piles, drilled shafts, abutments, wingwalls, and pier caps. Most of these elements will be conventionally reinforced concrete, where steel rebar is used to provide tensile strength, shear resistance, and flexure capacity. Driven piles will often be comprised of prestressed concrete, where steel tendons are placed into tension at the time of casting so that the concrete in the section remains in constant compression. These elements all generally have the same durability considerations.

There are two primary durability concerns for reinforced concrete elements: corrosion of the reinforcing steel, and degradation of the cover concrete. In healthy, intact concrete, reinforcement corrosion is inhibited by an oxide layer formed on the outside of the steel rebar. This oxide layer is formed when steel reacts with sodium and potassium hydroxide naturally present during the hydration of cement (Bertolini et al. 2013). The layer is protected by the high pH and low permeability of the cover concrete. If the concrete cover becomes less effective or allows chlorides to penetrate to the steel, the oxide layer can be broken down and the steel can begin to corrode. Since steel expands as it corrodes, even a small amount of corrosion can cause cracking or spalling of the cover concrete, making the reinforcement even more susceptible to corrosion. While the primary issue is the loss of reinforcing steel, it is important to determine and understand the underlying causes of corrosion initiation or cover degradation to assess the durability of concrete elements. Figure 50 presents a diagram showing causes of concrete degradation and reinforcement corrosion based on the work by Bertolini et al. (2013).

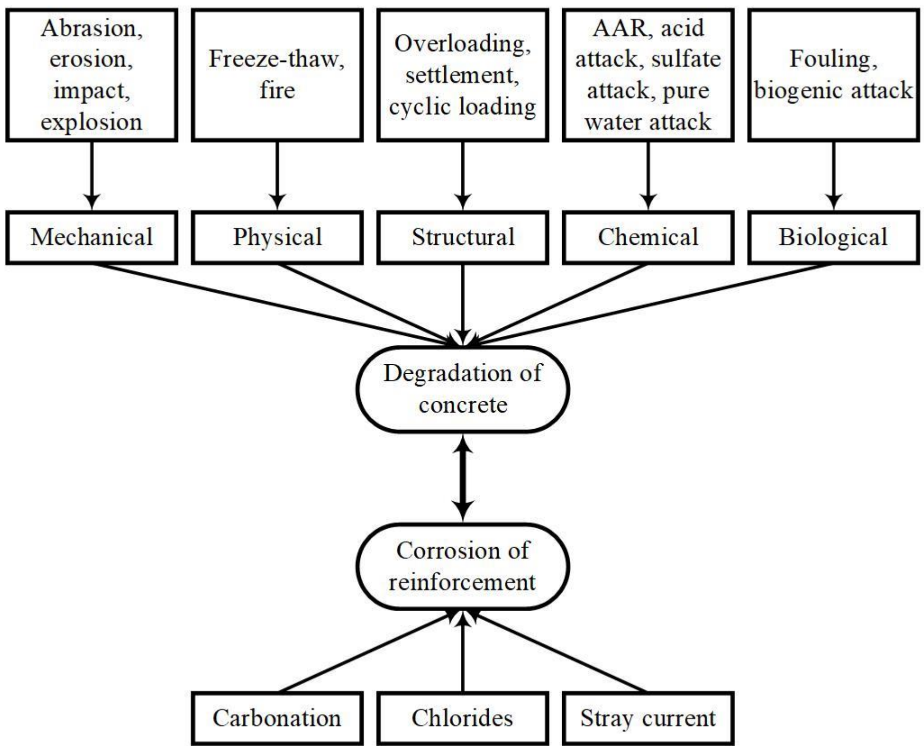

Figure 50. Illustration. Causes of deterioration of reinforced concrete structures (Adapted from Bertolini et al. 2013)

One of the most common underlying causes of reinforcement corrosion is cracking of the concrete cover. The cracking can be initiated by several causes: shrinkage/drying, flexural cracking (in tensile areas), shear cracking, temperature fluctuations, freeze-thaw cycling, impacts/damage, or poor initial quality. While the cracking itself often does not greatly impact the capacity of the element, it allows for ingress of water, chlorides, and contaminants that initiate corrosion in the reinforcement steel. Chemical attacks from acids, sulfates, pure water, or other ions can degrade the concrete cover, potentially exposing reinforcement. Abrasion and erosion by water or blowing particles can also wear away concrete cover, eventually exposing rebar. For unreinforced concrete, the durability would be controlled by how much concrete section loss is acceptable.

Chlorides

Even without cracking, chlorides can ingress through the cover concrete and initiate corrosion in the reinforcing steel. Significant work has been performed by Bazant (1979), Vassie (1987), Sohanghpurwala (2006), Liu (1996), Thompson et al. (2012), ACI (2016), Weyers (1998), and others to assess how quickly chloride ions diffuse through concrete as well as what concentration is required to initiate corrosion. The source of chloride ions can be from direct contact with seawater, exposure to roadway deicing runoff, exposure to saltwater laden air, chloride contamination of the initial concrete, or exposure to soils with high concentrations of chloride or other ions. Some chlorides may have been present in the initial concrete and are bound in the cement paste. These chlorides are not commonly believed to contribute to steel corrosion (ACI 222R 2001), although test procedures do not always distinguish between bound and unbound chlorides. Unbound chlorides generally accumulate on the surface due to environmental exposure, and then diffuse into the concrete toward the reinforcement. Corrosion can be initiated when the chloride concentration reaches a threshold amount at the rebar.

Carbonation

Carbonation refers to the conversion of calcium hydroxide ($Ca(OH)_2$) present in cement reacts with carbon dioxide (CO_2) to form calcium carbonate ($CaCO_3$). As this process occurs the hydroxide level decreases and the pH is reduced from a range of 11.0 to 13.0 for fresh concrete to a pH of as low as 7.0 for fully carbonated concrete (Mehta and Monteiro 2014). As the source of CO_2 is generally the exterior air, carbonation occurs as a front that works inwards from all exterior faces of the element. When the carbonation front reaches the depth of the reinforcement steel, the oxide layer passivates, and corrosion can begin. Carbonation of concrete at the rebar level can lower the chloride threshold where corrosion begins. Since water is required in the carbonation reaction and corrosion is aided by the presence of moisture and ions that reduce electrical resistance, carbonation, chloride penetration, and humidity can combine to initiate corrosion, as shown in figure 51.

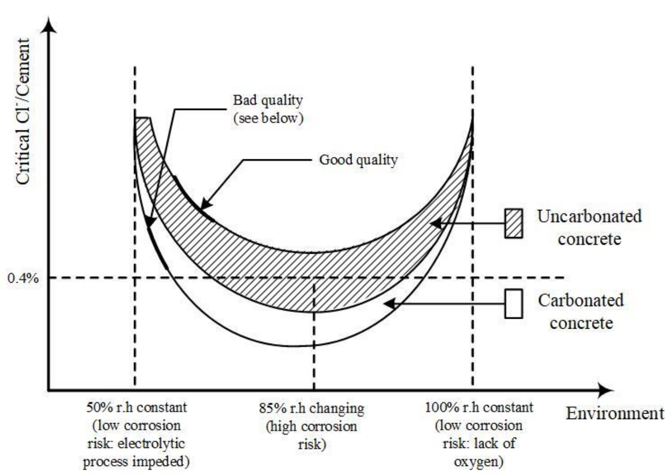

Figure 51. Illustration. Overlapping impacts of carbonation, chlorides and humidity

Freeze/Thaw Cycling

Freeze thaw cycling can lead to a reduction in concrete strength (in affected areas) over time and eventual exposure of reinforcement. This issue is mostly evaluated in the integrity assessment, where existing damage is observed, and petrographic analysis determines how susceptible the mixture is to freeze-thaw cycling. The susceptibility of concrete to freeze thaw cycling is primarily dependent on: air entrainment (shape and amount), and the water to cement ratio of the concrete paste. Most of the time, the integrity analysis will provide a yes/no type answer: the concrete is undergoing significant freeze-thaw damage and needs to be protected or replaced, or that concrete has not experienced freeze-thaw damage and has a location-appropriate mixture that will resist future freeze-thaw action.

Steel Elements

The primary issue affecting the service life of steel elements is the conversion of steel to iron oxide, referred to as rusting or corrosion. All steel elements will rust over time, depending on the exposure to both humidity and oxygen. This corrosion can be prevented using coatings like paint, rust itself (in the case of weathering steel), and coal tar epoxy. Some State codes (TxDOT 2014; CalTrans 2015) prescribe the use of a sacrificial thickness of steel to account for future deterioration. Comparing the prescribed sacrificial thickness with the observed corrosion allows comparison of the rate of corrosion observed at the bridge site.

Cathodic protection systems are commonly installed as a preventative or remedial measure to prevent corrosion from occurring or spreading by changing the electric potential of the element. Cathodic protection (CP) systems often have a limited lifespan that can impact the longevity of elements with CP systems installed previously or during the reuse process.

Coating Health

Many steel elements are coated with paint, coal tar epoxy, or another sealer whose intactness impacts the durability assessment. Minor chipping or pitting of the coating may occur during an investigation, which can be repaired as necessary. Even if significant corrosion has not occurred over the lifespan of a steel element, failure of this coating can accelerate corrosion, ultimately limiting the remaining service life of the element. Replacement of a damaged coating may be impossible or prohibitively expensive.

Timber Elements

The primary issue impacting the service life of timber elements is biological attack from boring insects or fungi. The types of insects and fungi that a timber element is susceptible to are highly specific to regions, type of timber, and preservatives applied to the timber. Since preservatives are applied to timber elements externally, it is common for the elements to begin to decay from the inside with the decay working its way outward. Since the interior of timber elements generally lacks the protective preservative, fungi and insects can cause a rapid loss in element strength once they begin to take hold.

Masonry Elements

The durability of masonry elements is primarily controlled by the condition of the mortar (grout) binding the masonry stones or blocks together. The mortar can become weathered from exterior forces such as wind, rain, or other particulate matter. Thermal expansion and contraction forces, shrinkage, or freeze-thaw cycling can cause cracking that can increase the mortar's susceptibility to additional erosion or plant growth. Plant growth inside of cracked mortar will lead to additional cracking and potential spalling of portions of mortar. Loss of mortar can lead to long term reductions in overall strength, as well as shifting of blocks.

Over time, the masonry blocks themselves can also become weathered by exterior forces such as wind, rain, and particulates. This erosion is generally slow and the impact of it on the strength of the masonry element is generally minimal. Shifting blocks due to settlement, grout loss, or previous seismic loading can lead to cracking within the blocks themselves. If this cracking is extensive, further analysis of the blocks may be necessary to determine if the cracks have caused reductions to the section strength.

PRELIMINARY EVALUATION

The purpose of the preliminary evaluation is to identify previous performance issues, adverse environmental conditions, and common concerns that may impact the durability of the bridge substructure. There is a large amount of overlap between the preliminary evaluation, the data collection in Chapter 3, and the integrity assessment in Chapter 4. Understanding the environmental conditions at the foundation and their potential durability impacts will assist in devising and implementing an appropriate test plan. Environmental considerations that warrant special consideration include chloride/ion exposure, average humidity, polluted water, aggressive soils, abrasive winds/currents, and freeze/thaw cycling. An overview of the major areas of the preliminary evaluation for all structure types are given in table 22.

Table 22. Steps to the preliminary evaluation and the outcomes of that investigation

Evaluation Procedure	Reason/Outcomes
Records Review	Review of past inspection history allows for assessment of the time history of bridge performance
Environmental Conditions	Environmental conditions at the bridge dictate the types of deterioration expected. Important aspects to consider are: exposure to deicing salt, exposure to salt water, fresh water, contaminated soil or water, humidity, stray currents, or freeze/thaw conditions
Visual/Physical Survey	Document extent of cracking, signs of rust staining or efflorescence, erosion of concrete paste, and extent of spalling. Locate delaminated areas using hammer sounding and physical methods. Generally, overlaps with integrity assessment in finding the current condition of the concrete.

Deterioration of the substructure can be quantified in a high-level manner during this phase of the investigation. Broad categories, such as percentage area cracked, damaged, corroded, etc. can help estimate what types of repair will be needed, how extensive these repairs will need to be, and what forms of testing are required. This step can be used to determine the feasibility of reuse prior to an extensive testing program.

Concrete Elements

Records Review

The prior history of durability related issues is an important consideration for concrete elements where reuse is being investigated. Concrete foundations with a history of corrosion related issues will likely require substantial rehabilitation prior to reuse. If cracking or spalling is widespread, it is likely that measures such as encasement or cover replacement will be necessary. When cracking or spalling is less extensive, cheaper repair options, such as wrapping by FRP or patching may be available, depending on a thorough analysis of the concrete system.

Environmental Concerns

The susceptibility of concrete elements to corrosion and carbonation is highly dependent on their surrounding environment. The most important environmental considerations are: exposure to chlorides, exposure to water, aggressive soil conditions, and ions, freeze-thaw cycling, exposure to CO_2, and humidity. For many foundations, multiple of these conditions may be present simultaneously.

Concrete elements in bodies of saltwater are the most vulnerable at the waterline, where there is ample access to both moisture and oxygen. Areas below the water can be susceptible to corrosion, although the lower availability of oxygen helps impede the corrosion process. Areas that are in alternatively wet and dry environments are highly susceptible to corrosion. Foundations can also become exposed to chloride ions from deicing runoff, chloride laden air, or soils high in chlorides.

Concrete elements exposed to bodies of fresh water can also suffer from reinforcement corrosion or concrete deterioration. Constant exposure to water lowers the electrical resistivity of concrete, creates greater opportunities for freeze-thaw action, and increases carbonation potential. Like saltwater, corrosion initiation related to the water is most likely near the water line or in alternatively wet/dry areas. Water that is acidic or low in calcium ions can lead to accelerated deterioration of cover concrete. Water that is high in ammonium, magnesium, and sulfate ions can lead to more rapid degradation of submerged concrete. Concrete elements in soil containing ammonium, magnesium, or sulfate ions can also suffer ion-induced deterioration.

Areas exposed to roadway runoff or saltwater can experience chloride intrusion that reduces their lifespan. If necessary, sampling of fresh water or soil can be performed to determine if corrosive chemicals or ions impacting the substructure are present. Identifying these potential issues during the preliminary evaluation allows for a better thought out, more streamlined durability assessment.

Carbonation is caused by the reaction of CO_2 with the alkaline components of the cement paste. This reaction occurs due to the presence of both CO_2 and humidity. While CO_2 is present in low concentrations in air, the presence of nearby industry, heavy traffic, or other combustion sources can elevate the CO_2 levels. Areas with higher average humidity will be more susceptible to carbonation, as water is part of the chemical process.

Initial Inspection

The initial field investigation generally consists of a thorough visual and physical inspection, documenting the full of extent of cracking, spalling, and delamination. If recent routine inspection data is available, a testing plan can be devised to allow testing to be performed alongside the inspection.

Steel Elements

Records Review

The important quantities to consider while reviewing records of steel elements are the signs of rusting, coating failures, drip spots, and prior repairs. The history of previous rusting of elements is important as it helps establish the timeline of observed corrosion issues at the bridge. Documentation of observable corrosion over time can also establish how the steel coatings performed over time.

Environmental Concerns

The most important environmental factors for steel elements are their exposure to moisture, ions, and oxygen. Steel elements located in dry environments with low exposure to ions can have a virtually infinite service life. Direct access to water is not required for rust initiation, as the moisture available in air can initiate and propagate corrosion. Steel elements can become exposed to high levels of ions through exposure to saltwater, saltwater spray, ions present in the soil, or through saltwater laden air near the coast. Submerged steel elements are generally less prone to corrosion than the portions in splash zones or alternatingly wet environments. Underground steel elements that are below the groundwater table are not generally susceptible to corrosion as there is too little dissolved oxygen for the corrosion process to initiate. Corrosion of steel piling will most often occur near the ground surface, especially when above the water table.

Steel elements placed in bodies of water require careful consideration of their environmental conditions. The specific nature of an aquatic environment greatly impacts the corrosion rate of steel elements exposed to it. Corroding steel acts as an anode, donating electrons to the surrounding environment. Steel elements exposed to saltwater, brackish water, or water high in ions will be a more effective anode and corrode at an increased rate. As steel corrodes, the rust can form a layer on the outside that slows down additional corrosion. High velocity water will erode this layer and expose fresh steel, increasing the corrosion rate. Various chemicals and pollutants can increase the rate at which steel in aquatic environments corrodes. Exposure to stray electrical currents can lead to localized severe corrosion. Acidic environments will accelerate the rate of corrosion. Microbial life can coat the outside of steel elements, either slowing or greatly increasing corrosion.

Initial Inspection

Visual and physical inspection are standard inexpensive measures that can greatly inform the reuse investigation. This inspection documents the extent of deterioration and measures the extent of section loss, where rust is encountered. Since the most commonly reused steel substructure elements are piles, there is frequently little above ground portion to be analyzed.

Timber Elements

Records Review

Records for timber elements include documentation of previous field testing, history of decay-related issues, and repair history. Previously performed field testing can consist of core sampling, hammer sounding, resistographs, and NDT. The history of deterioration issues and related repairs can be used as an initial gauge on how durable these elements have been.

Environmental Concerns

The susceptibility of timber elements to decay is directly related to their exposure to moisture and oxygen. Timber elements that are either underwater or beneath the ground water table can have virtually indefinite service life, mainly due to the lack of oxygen present when submerged (Hannigan 2006). Timber elements in alternatively wet and dry environments are prone to decay regardless of the salinity of the water present. Piles in dry environments can have long service lives, although moisture from ambient humidity can allow fungi to invade. Most insect borers cannot attack submerged piles, although when present they can attack dry piles or piles in alternatively wet/dry environments.

Initial Inspections

The initial inspections performed on timber elements generally consist of visual and physical inspection performed on exposed sections. For timber pile bents, the timber pile just above the ground or water surface, or in the intertidal zone is often the most prone to deterioration. Routine inspection consisting of visual inspection and hammer sounding can establish whether the interior of the pile has begun to deteriorate. Exterior damage, brooming, and other signs that the exterior coating has been damaged can be identified visually. Timber piles framed into a pile cap will generally be unobservable without excavation. During the integrity evaluation, the current state of timber pile rot, damage from marine borers, and other damage is evaluated to sufficiently establish the current conditions of the element. Previous repairs including encasement, FRP wrapping, and PVC wrapping are all typically inspected for deterioration. The primary considerations include establishing that the encasement is still watertight and that the repairs sufficiently cover all vulnerable portions of the elements.

Masonry Elements

Records Review

Important records for masonry elements include the inspection history and observations of cracks, and their growth over time. This information can be used to determine if crack growth is ongoing or has previously stopped. These records may include enough information to determine if there is differential settlement or movement driving the crack growth.

Environmental Concerns

The primary environmental concerns for masonry structures are their exposure to water, humidity, deck runoff, and freeze-thaw cycles. The presence of moisture or water is necessary for either freeze-thaw cycling or plant growth to be a concern. Both issues can create cracking in the mortar joints between blocks. In dry environments, the major long-term issue remaining that can impact masonry substructures is abrasion/erosion of the face of the element and settlement-induced cracking and shifting of blocks.

Initial Inspections

The testing methods employed for masonry elements are primarily focused on establishing the current conditions and are largely discussed in Chapter 4. Visual and physical inspection can be used to determine the health of the mortar joints and the masonry blocks. Mortar joints are examined visually for cracking and deterioration, then probing and brushing is used to determine if portions of mortar have become loose and dislodged. Inspection of the blocks can identify cracking and weathering on the exterior surface.

FIELD MEASUREMENTS AND TESTING

A test plan can be developed following the concerns raised from observations, local experience, and environmental considerations. A high-quality test plan would account for inherent variability in the foundation system condition as well as potential variance across the site. For example, chloride exposure may not affect all piers equally, and chloride concentrations will likely be variable even across an element. Piers exposed to runoff of deicing chemicals through joints in the bridge deck will generally have greater chloride exposure than other piers not exposed to runoff. Timber and steel piles will be most prone to deterioration where portions are above or at the groundwater table, and not all piles will decay at the same rate.

Concrete Elements

The findings from the preliminary evaluation can be used to develop a test plan for concrete elements that addresses the identified issues. Examples of the types of tests (not including those mentioned during integrity testing as in Chapter 4) that are typically performed in this phase are given in table 23.

Table 23. Durability related testing for concrete elements

Available Testing	Issue identified during preliminary evaluation	Notes
Cover Measurement	Corrosion, chloride exposure, carbonation	Determine cover thickness important to evaluation of other durability issues.
Chloride Testing	Exposure to chlorides	Determine profile of chloride diffusion into cover concrete. Initial chloride testing can be limited to surface and depth samples, to ascertain the magnitude of bound and unbound chlorides
pH testing	Carbonation	Perform pH testing on extracted cores to determine depth of carbonation penetration
Half-cell potentials	Active corrosion	Perform half-cell potential testing in areas of suspected corrosion
Electrical Resistivity	Potential for corrosion	Useful for finding areas of corrosion or areas susceptible to corrosion

A variety of testing methods are available for concrete elements, ranging from simple visual and physical inspection to NDE/NDT, to sample removal and testing. In general, durability-related tests on concrete members seek to determine the extent of cracking present, the presence and rate of reinforcement corrosion, the extent of carbonation, and the extent of chloride ingress into the cover concrete.

Cover Depth

Knowledge of the cover depth with reasonable accuracy is crucial when evaluating susceptibility to chlorides, carbonation, erosion, or other forms of cover degradation. The thickness of the concrete cover can be estimated from design plans, although there is often a fair amount of variability between the design thickness and the actual thickness. Covermeters and Ground Penetrating Radar (GPR) are commonly employed to determine the depth and placement of rebar. Covermeters use eddy current detection to find rebar from the disturbances in the magnetic field, while GPR relies on radar reflections. Covermeters are less affected by the presence of voids, moisture and other environmental effects, while GPR can provide more coverage and can be used for more closely spaced rebar. The depth to rebar is variable across single concrete elements, and having many measurements provides more confidence in the minimum cover depth and allows for estimation of the cover depth variability.

Cracking and Cover Health

Cracking of the cover concrete is primarily assessed by recording cracks lengths, widths, and locations during visual inspections. Repeated inspections allow for determination of how cracking has evolved over time. Signs of efflorescence indicate that water can freely flow through cover concrete cracks. Likewise, signs of rust staining indicate that rust is occurring and being brought to the surface.

Delamination surveys are commonly performed on bridge decks by dragging a chain across the deck and listening for hollow sounding areas that indicate the presence of delamination. ASTM Standard D4580 (ASTM 2012e) provides the methodology for this testing. On bridge piers and other concrete elements, the delamination survey can be performed by sounding a hammer or steel rod off the surface of the concrete. The same distinctive hollow sound is produced when a delaminated area is found. Impact-echo and ultrasonic pulse response methods have also commonly been used to locate delamination of cover concrete. Ground Penetrating Radar (GPR) uses radar reflections to locate internal concrete voids and is commonly used on bridge decks. For bridge piers, wall climbing robots have been investigated. Infrared thermography detects delamination and voids by measuring altered heat flow in these areas.

The presence of delaminations, spalling, or cracking is an indication that corrosion is actively occurring. Performing half-cell potential tests allows the engineer to determine the locations that are currently experiencing corrosion. Some areas of active corrosion may not yet show significant damage to the exterior layer of concrete. Areas of active corrosion identified during half-cell potential testing will likely require some form of repair and replacement to end active corrosion. Prior to conducting half-cell potential measurements, it is necessary to establish a path of electrical conductance.

Another measurable aspect of the health of cover concrete is the porosity of the concrete. More porous concrete will allow for faster ingress of chloride ions and quicker progression of the carbonation front. The 90-day ponding test (AASHTO-T-259) and the electrical method for determining the resistance to chloride ingress in ASTM Standard C1202 (ASTM 2012f) provide methods for evaluating the concrete's ability to resist chloride penetration. The air permeability test given in SHRP-S-329 allows for determination of the concrete's ability to resist CO_2 ingress, and therefore carbonation susceptibility.

Corrosion

The most direct method of measuring active reinforcement corrosion is using half-cell potential testing. Half-cell potential testing, described by ASTM Standard C876 (ASTM 2015c), measures the difference in electrochemical potential between an area of concrete and a ground connected the reinforcement in that area. A commonly used reference electrode for this testing is copper-copper sulfate. ASTM Standard C876 (ASTM 2015c) provides the following breakdown (table 24) of how measured potential translates to corrosion likelihood:

Table 24. Measured Half-Cell potential vs. likelihood of corrosion

Measured Half-Cell Potential	Likelihood of Corrosion
Potential > -200 mV	90% chance of there no corrosion occurring
-200mV > Potential > -350mV	Increasing likelihood of corrosion
-350 mV < Potential	90% of there being ongoing corrosion

While the values shown in table 24 provide some indication to the likelihood of corrosion, given a measured potential, the half-cell potential measurement is the most effective approach when used as a comparative tool. Areas with potentials higher than the surrounding areas are the most likely to be undergoing corrosion, and sharp differences between nearby areas indicate that a corrosion cell has been formed. A positive reading of potential is an indication that that rebar is acting as a cathode, meaning another portion of the element is likely undergoing corrosion.

Electrical resistivity tests, as documented by Mehta and Monteiro (2014) and Bertolini et al. (2013), apply an electrical current to the concrete and measure the electrical resistance provided by the concrete. This testing is often performed in-field using two electrodes, one attached to the concrete face, and another attached to rebar that is electorally contiguous with the section being tested. High resistances over 200 Ω.m indicate that the likely corrosion rate is negligible, and that future corrosion is unlikely if the concrete cover remains intact. As a generality, concrete will be much more conductive when wet than when dry, so the moisture content at the time of testing will heavily impact electrical resistivity measurements. Bertolini et al. (2013) provide the following criteria (table 25) for interpreting resistivity measurements:

Table 25. Criteria for interpreting resistivity measurements in concrete

Concrete Resistivity (Ωm)	Corrosion Rate
> 1000 Ωm	Negligible
> 500 Ωm	Low
200-500 Ωm	Modest
100-200 Ωm	High
<100 Ωm	Very High

Chloride Ingress

Chloride testing is typically performed on powder samples extracted by drilling from various locations on the pier. Chloride testing can also be performed on core samples taken for compression testing or petrographic analysis. By obtaining chloride samples from multiple depths at each location, a profile of chloride concentrations can be established. In general, the highest concentrations of chlorides will be at the surface of the element. The chloride concentration will drop to the background level in the original mixture within several inches of the face. As chlorides enter the concrete from the surface, they diffuse toward the interior of the element, following Fick's law. The coefficients for a Fick's Law model can be determined from chloride testing at multiple depths following ASTM Standard C1152 (ASTM 2012g).

Two forms of chloride testing are traditionally applied to concrete bridge elements: the acid-soluble test in ASTM Standard C1152 (ASTM 2012g), and the water-soluble test in ASTM Standard C1218 (ASTM 2015d). The primary difference between the two test methods is that the water-soluble test provides the amount of free chloride ions in the concrete pores and trapped water, while the acid soluble test frees the bound chloride ions from the cement paste and aggregate. The results from the acid-soluble test include both the bound and free chloride ions. While there is debate over whether the total (acid-soluble) or unbound (water soluble) chloride concentration is more important, the acid-soluble test generally produces more consistent results and is more favored by researchers (Sohanghpurwala 2006). Field methods for performing the acid-soluble test are also available and provide usable results with much more rapid testing than traditional laboratory testing. By performing an acid soluble test beyond the depth of chloride penetration, the amount of bound chlorides in the concrete can be determined and subtracted from the total chloride concentration obtained at shallow depths affected by chloride ingress.

Carbonation

Carbonation will manifest itself as a zone of lower pH near the surface, and the carbonation front will ingress into the concrete over time. If carbonation is found to be shallow or insignificant from limited testing, it can often be assumed to be inconsequential and disregarded in the life cycle analysis. If significant depths of carbonation are noted, service life modeling can be performed to estimate the penetration rate of carbonation. If the carbonation front reaches any layer of reinforcement, corrosion can begin, leading to cracking and cover damage that can increase corrosion rates.

The most commonly used testing protocol for determining the extent of concrete carbonation is the use of Phenolphthalein. Phenolphthalein is pH indicator which turns magenta when exposed to concrete with a pH above 9.0. When applied directly to removed cores of concrete elements, the portions of concrete that do not turn the phenolphthalein magenta are considered carbonated. This test, when performed on a core removed from the face of a concrete element, allows the determination of the depth of the carbonation front.

Freeze/Thaw

Petrography performed during the integrity analysis can be used to confirm the mixture properties, including entrained air, size of entrained air bubbles, and water/cement ratio. Existing freeze/thaw damage can be observed using petrography. Temperature measurements taken at the elements in question or local weather stations can be used to better estimate the amount of past and present freeze thaw cycling at the element.

Steel Elements

Since much of the evaluation of steel elements is through visual and physical inspection, the major components of the field evaluation program revolve around underwater and underground inspection of elements. Underground inspection can be performed through excavation of test pits and direct exposure of the steel piles being considered. If care is not taken during this step, the foundation may be undermined or damaged by the excavation process.

When investigating corrosion on underwater elements, the most critical portions are often at the water line or just below the concrete pier cap. To establish how much section loss has occurred, it is important to clean the rust products off a representative portion. This portion can then be measured to assess the remaining section. The corrosion rate can also be established by comparing the current remaining steel with the initial section size, and accounting for how long the element has been in service. As bridges undergo regular inspection cycles, multiple measurements of the corrosion extent over time are likely to be recorded. Comparing the corrosion extent over time will help the bridge reuse design ascertain whether the corrosion rate has constant, increasing, or decreasing.

Available Testing Methods

Testing on steel elements for durability issues generally revolves around determining the extent of corrosion, the rate of corrosion, and the extent of fatigue cracking. The extent of corrosion is found through visual and physical inspection using calipers, ultrasonic thickness gauges, or other measurement devices. The current section dimensions are found by removing all rust and debris and measuring the remaining bare steel. For underground elements, even simple observations can be difficult to obtain, requiring the installation of test pits to expose the element. This testing may not be possible in all situations and can generally only be performed on a select number of representative piles.

The primary drivers for corrosion in underground locations are: areas of low pH, high sulfate or chloride content, alternating wet/dry cycles, and differential soil layers that promote the creation of macrocells. If aggressive soil conditions are expected, the appropriate evaluation procedure for these environments is given in AASHTO R27-01 (2010).

Timber Elements

Available Testing Methods

Coring, hammer sounding, and various destructive and nondestructive testing methods are commonly employed to determine the current condition of timber elements. By in large, this testing is covered in the integrity evaluation discussed in Chapter 4. Also discussed in Chapter 4 is the monitoring of ground water, which an important consideration into the longevity of timber elements. In general, timber elements that are permanently below the water surface or the ground water table are not susceptible to decay or insect boring.

The most widely used testing related to timber elements is coring with a small hand drilled tool. An experienced inspector can determine the resistance provided by the pile and establish when rotted or deteriorated sections are present. Coring tools are inserted in a similar manner but allow the extraction of a small plug of material. This extracted slug can be inspected to determine if decay has occurred and where in the section that decay has occurred.

Determining the moisture content of timber elements can help determine how prone to decay the element is. Since decaying elements have higher porosity and water content, a very high moisture content can indicate that decay is currently happening. One common method of determining the moisture content is to drive electrodes into the element and measure the electric conductivity of the element. The probes are driven approximately 2.5 inches (64 mm) into the element, and a current is passed through the element to measure the electric conductivity of the element.

Masonry Elements

The field measurements and testing phase of masonry inspection is largely covered during the integrity assessment. Issues that will impact the durability evaluation as well primarily consist of grout deterioration and possible corrosion or steel rebar, ties or anchors (for reinforced masonry sections). Freeze-thaw related deterioration can potentially impact the masonry blocks themselves when particularly serious. Grout deterioration can consist of plant growth on exterior portions, excessive permeability, or freeze-thaw action. Testing of the permeability of the grout can be performed to determine its permeability to air and liquids.

SERVICE LIFE PREDICTION AND LIFE CYCLE COSTS

Service life prediction is the final aspect of durability assessment for bridge substructures being considered for reuse. Service life prediction is often performed for multiple design alternatives, accounting for any repair work, encasement, or protection systems installed on the reused substructure element. The life cycle costs can include projected repair costs, replacement costs, deconstruction costs, and other maintenance costs associated with the existing foundation and any reuse alternatives. The specific issues to consider for various material types are provided below.

Concrete Elements

The major aspects that influence the service life of concrete elements are: chloride-related corrosion, carbonation-related corrosion, freeze-thaw damage accumulation, and surface erosion. These issues do not immediately have a large impact on the capacity of concrete elements, but their effects can accumulate over time to lead to excessive damage, most often in the form of reinforcement steel corrosion and damage to the cover concrete. After the onset of corrosion in the steel, the section can rapidly lose capacity and functionality, as corrosion and section loss will form a feedback loop of damage.

Chloride Related Corrosion

The service life of a concrete element exposed to chloride is defined by the length of time until corrosion initiation and the length of time required for corrosion to cause cracking. As chlorides enter through the surface of the concrete, they diffuse down toward the reinforcement level. The rebar remains in a passive state and does not corrode until a threshold concentration of chlorides is reached. The exact concentration that initiates corrosion varies depending on details of the concrete mixture and other conditions, but ACI committee 201 (2016) recommends a chloride threshold of 0.2 percent of the weight of cement. Other researchers, such as NCHRP 558

(Sohanghpurwala 2006) have described the chloride threshold as a probabilistic threshold, where increasing chloride content increases the likelihood of corrosion initiation. After corrosion is initiated, the rebar begins to corrode until a critical cracking force is reached. Bazant (1979) proposed early models that included estimations for both the initiation time for corrosion and the propagation time.

NCHRP 558 (Sohanghpurwala 2006) provides a methodology for estimating the remaining service life of concrete elements exposed to chlorides. The methodology assumes the cover depth, diffusion coefficient, and surface chloride concentration are random variables. The concrete surface is viewed in finite elements, where each element's cover depth, surface chloride concentration, and diffusion coefficient are random variables from the pertinent probability distribution. The parameters for the surface chloride content are found by looking at the average and standard deviation of all surface chloride samples. The surface chloride concentration can be assumed to be to constant or linearly increasing over time. A single chloride threshold is found for the entire model by fitting the damage predicted by the model to damage observed in real life. For example, if 10 percent of the concrete surface had spalled or delaminated at an age of 40 years, a chloride threshold would be chosen so that the model predicts 10 percent damage at that age. Repairs are accounted for by eliminating that damage and continuing the model for additional life. The time to cracking after corrosion is initiated is assumed to be five years, although this is a variable quantity. Epoxy-coated rebar is accounted for by assuming a probability that the epoxy barrier is damaged. The chance of rebar damage increases as the age of the section increases.

Various software packages have been developed to predict the durability and lifespan of concrete elements. Life-365™, developed by a consortium consisting of W.R. Grace Construction Products, Master Builders, and the Silica Fume Association, was developed to help engineers predict the remaining service life of chloride exposed concrete elements, as well as the future life-cycle costs. Another software package currently used to estimate the lifespan of concrete elements is STADIUM®, developed by SIMCO. Both software packages can estimate the propagation time and corrosion initiation time given various environmental conditions.

Carbonation Related Corrosion

Carbonation occurs from the outside inward as the concrete is exposed to both carbon dioxide and humidity. From the cores extracted, the carbonation penetration depth can be measured. The simplest method for determining the carbonation penetration is by applying phenolphthalein to extracted cores and measuring the portions with a pH below 9.0. The portions with the lower pH are the carbonation penetration depth. Once the carbonation depth is known, a simple model can be fit to the carbonation penetration, following Eq.(25).

$$x = kt^n \tag{25}$$

where x is carbonation depth, k is a constant, t is time to measurement, and n is the time factor, generally taken as 0.5 (Bertolini et al. 2013)

By knowing the carbonation depth, x from measurements, and the time, t the constant k can be solved for. From this, the future carbonation penetration can be ascertained. Unlike chloride modeling, the modeling of carbonation penetration is usually not as sophisticated and does not include a method for determining the percent of the element that will damage over time. Given that carbonation will also interact with chloride penetration, it is good practice to assume the end of life for concrete elements is simply the carbonation penetration time plus the time from corrosion initiation to cracking. If the carbonation penetration has been relatively non-existent, then service life modeling with respect to carbonation is likely not necessary.

Freeze-Thaw Damage

The primary knowledge required to assess the freeze-thaw durability will come from petrographic analysis performed during the integrity assessment. The important takeaways are the w/c ratio and the level of air-entrainment. Higher w/c ratio concrete will be highly frost susceptible, but concrete in the typical range of around 0.4 to 0.5 w/c ratios will generally perform well given the proper air entrainment. The proper air entrainment can be confirmed by petrographic analysis, and generally requires entrained air be in the correct size bubbles and relatively closely spaced. Bertolini et al. (2013) suggest a total air entrainment around 4 percent to 7 percent, with the maximum spacing between bubbles being approximately 0.1mm to 0.2mm. Ideally, the entrained air will be in small bubbles on the order of 0.002 inch (0.05 mm) to 0.04 inch (1 mm) in diameter. Figure 52 shows data gathered by the U.S. Department of the Interior (1975) that compares the durability factors (as functions of water to cement (w/c) ratio) for non-air-entrained concrete and concrete with 4 percent air entrainment. The durability factor shows the relative modulus of concrete after a set amount of freeze-thaw cycles (300 for this data).

Currently, there is not a defined service life prediction technique for freeze-thaw action, so bridge managers are left to weigh the amount of freeze-thaw damage observed during petrography with the appropriateness of the mix design to determine if the concrete cover will perform adequately. Existing concrete can be defended from future freeze-thaw action through encasement, assuming the encasement is thick enough and has the appropriate properties.

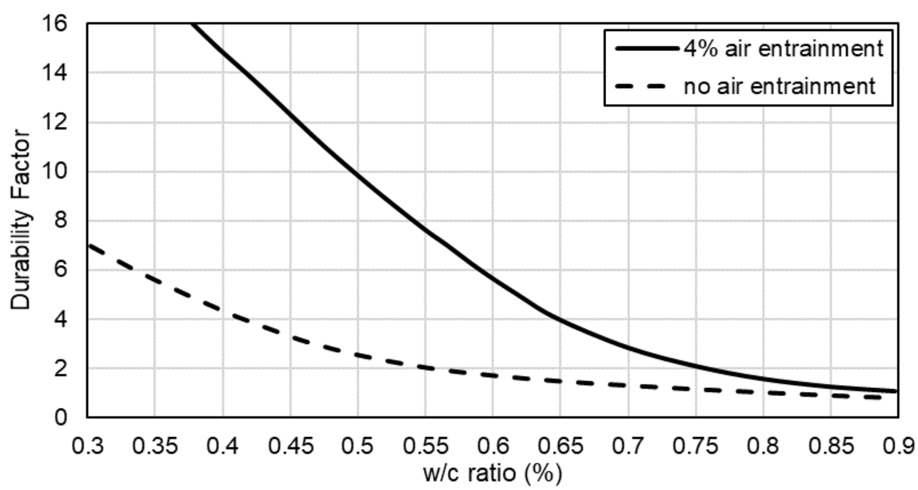

Figure 52. Graph. Durability vs. w/c ratio

Surface Erosion

The surface erosion of concrete will not generally be an issue until the cover protecting the rebar has worn away or has become very thin. The rate of surface erosion can be estimated when sufficient data is available from prior observations. If paste erosion continues, the rebar may become exposed faster than expected. The loss in cover concrete can lead to increased chloride or carbonation penetration at steel level, or direct exposure of the steel in severe enough cases

Coatings

Several different coatings are available for concrete elements, including hydrophobic coatings to reduce chloride ingress, and hydrophobic coating to reduce moisture content and increase resistivity. Concrete elements can also be wrapped in PVC or Fiber Reinforced Polymers (FRPs). These coatings and wrappings can provide large advantages to substructures with light chloride, freeze-thaw, or carbonation exposure as they over time reduce the amount of moisture within the section. The primary damage to existing coatings will often be from external abrasion, cracking or impacts on the concrete surface. These coating and wrappings will often have a manufacturer specified lifespan that can be compared with the age of the bridge. Inspection of the coatings can be performed to ensure that there is no significant cracking, warping, or other damage.

Remaining Service Life

While the approaches discussed above can be used to predict the service life of concrete elements quantitatively, State DOTs frequently perform a number of tests or inspections to determine levels of deterioration to project remaining service life. Depending on the foundation reuse project, all or some of the following may be implemented to project remaining service life:

- Visual crack, delamination, and spall survey.
- Half-cell potential survey on accessible surfaces.
- Expose reinforcing steel for visual observation.
- Measure concrete cover.
- Collect cores and perform chloride profile analysis and carbonation testing.
- Conduct petrographic analysis.
- Perform unconfined compressive strength testing of cores.
- Collect soil and groundwater samples for corrosion potential evaluation (chlorides, sulfates, pH, and resistivity testing).

Reused foundations can also be monitored, either through visual inspections or instrumentations to carry out immediate repairs if any durability concerns arise during the service life of the reused foundation.

Life Cycle Costs

The life cycle cost analysis of concrete piers relies on the service life analysis discussed earlier as well as the remedial measures in place. Additional life cycle costs can originate from galvanic (sacrificial) anodes that need regular replacement or the electricity and regular maintenance required by impressed current anodes. Patching and repair of future spalls will also incur additional life cycle costs. Spalled and patched areas will be more likely see future damage, and the expected durability of these repairs, as discussed in Chapter 7, may be lower than the surrounding areas.

Steel Elements

Corrosion

Above ground steel elements are typically coated or given a sacrificial thickness of steel to allow for limited corrosion. This corrosion can be measured and monitored over time. The most important information when determining the expected amount of steel loss on above ground elements is from prior observations of those elements during the initial service life. AASHTO R27-01 (2010) provides a methodology for estimating the service life of underground steel piles in aggressive environments. Piles in fill or disturbed natural soils can conservatively be assumed to have a corrosion rate of 0.003 in/year (0.08 mm/year) (Hannigan et al. 2006). Browne et al. (2010a) provides a table that estimates future loss in steel element thickness in varying marine environments, shown in table 26.

Table 26. Loss in thickness (in) to steel elements exposed to fresh water or seawater (Browne et al. 2010[a])

Required Design Working life	5 yrs.	25 yrs.	50 yrs.	75 yrs.	100 yrs.
Common fresh water (river, ship canal, etc.) in the zone of high attack (water line)	0.006	0.022	0.035	0.045	0.055
Very polluted fresh water (sewage, industrial effluent, etc.) in the zone of high attack (water line)	0.012	0.051	0.090	0.130	0.170
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.022	0.075	0.148	0.220	0.300
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0.010	0.035	0.069	0.102	0.138

Coatings

Various coatings are available for steel piles and above ground elements (table 27). Typical coatings include paint, coal tar epoxy, or other epoxy type coatings. These coatings can be evaluated during inspection to ensure they remain in good health and continue to protect the element. If the coating breaks down, corrosion occur that limits the remaining service life. The total service life will then include the time to coating failure and the time until the sacrificial coating has been depleted. Reapplication of deteriorated steel coatings may to be performed on steel piles with deteriorated coatings, adding cost to their reuse.

Table 27. Available coating for steel piles and steel above ground elements

Coating Description	Period of Protection
Coal tar epoxy (15 mm to 20 mm thick)	10 – 20 years
Galvanizing (7 mm to 9 mm thick)	10 – 15 years
Metalized Aluminum	15 – 20 years
Concrete Encasement	25 years

Life Cycle Costs

The primary life cycle costs associated with steel elements will be for those elements with cathodic protection systems installed. Sacrificial anode systems will require continual replacement of the anode on a predictable schedule. Impressed current systems will incur electrical costs but will not need as frequent replacement. Both types of systems add complexity to the system and will likely require additional maintenance at some point during the service life of the bridge. Coated piles may also require service of the applied coating at regular intervals.

Timber Elements

Service Life

The service life of timber elements is mostly controlled by where they are installed and what preservatives were applied. Collin (2002) provides expected design life for timber piles given by their climate and exposure type, as shown in table 28. The given design lives for piles treated with a preservative appropriate for the geographic location of the element.

Table 28. Treated pile design life (Collin 2002)

Pile Exposure	**Pile Life Expectancy**
Permanently submerged	Indefinite
Fully embedded, treated piles with pile cap. Partially above groundwater	100 years or longer
Treated trestle piles over land	75 years in northern areas, 40 years in southern areas of United States
Treated piles in fresh water	Approx. 5 to 10 years less than land piles in same are
Treated piles in brackish water	Local experience necessary
Treated marine piles	50 years in northern climates, 25 years in southern climates

The results from the integrity evaluation will have a major impact on the remaining service life of timber elements. Since the primary factors influencing service life of timber are the health of the element and the environmental exposure, indications that deterioration has begun are indications that the remaining service life of the elements is limited. Even when decay, marine borers, and deterioration are observed on only some of the elements, this is often an indication that the other intact elements are nearing the end of their service life. Reuse of these piles will be completely dependent on the ability to cease the current decay, usually through some form of wrapping or encasement.

Preserved, Encased, and Wrapped Piles

An important test performed on timber elements during the testing phase is the measurement of preservative depth. Elements with insufficient preservative penetration may not have adequate protection against fungus and borers resulting in lower service lives than those listed in table 28. Wrappings like FRP, PVC, or other materials can be applied to timber elements to prevent to movement of water in and out of the element. Concrete encasement can both prevent the flow of water and provide strengthening for the section. A lower anticipated life of exterior timber protection may be acceptable in reuse situations although repairs to this protection may be needed during the bridge's lifespan.

Life Cycle Costs

The primary drivers of life cycle costs for timber elements will be related to additional inspection or future repair of encased or wrapped sections. Inspection costs will be regular and relatively predictable throughout the lifespan of the bridge. Repair costs may be more uncertain and will occur at an unknown place in time. The simplest analysis would place the year of replacement at the expected time of replacement. A more in-depth approach would be assign probabilities to several possible replacement timelines, and then assign a probability distribution to when the protection will need to be repaired or replaced. In general, the best approach is to ensure that the existing repairs will be sufficient for the entirety of the anticipated service life, negating the need to assess the life cycle costs associated with replacement and perform increased inspection.

Masonry Elements

Life Cycle Costs

The primary life cycle cost incurred by masonry elements will be in the form of plant growth removal, patching of mortar, and miscellaneous repair work. Masonry elements that contain steel reinforcement, ties, or anchors will have portions susceptible to corrosion that may increase the amount of monitoring or maintenance required.

CASE STUDIES

Milton Madison Bridge, between Milton, IN and Madison, KY

Initially, a visual and physical survey of Piers 5, 6, 7, 8, and 9 was conducted to assess the potential of reuse. Overall, the piers were experiencing varying amounts of cracking, delamination, spalling, efflorescence, and paste erosion. While some cracks were identified as being from initial construction, much of it was small and random in nature with minor efflorescence. Delaminated areas were found in original concrete in Pier 5, and some minor areas of delamination were found in repair patches on the remaining piers. Minor spalling had occurred along the edge of a vertical ledge and the face of Pier 6. Corroded reinforcement was visible near the water line at a cold joint that had eroded away on either side. The paste erosion observed was somewhat minor in nature, only affecting the outermost portions of concrete. A graphical condition survey was provided for Piers 5 through 8, showing the location of cracks and delaminations. The condition survey provided for the south face of Pier 5 is shown in figure 53.

Impulse Response (IR) was employed to locate delaminations (figure 54), and GPR was used to find the amount of cover concrete present and the depth to reinforcement. IR testing largely found the presence of delaminations, while the GPR testing found a cover depth between 1.5 inches (38 mm) and 6 inches (152 mm) thick. The cover depth was lower than the design cover depth in many locations. These tests are discussed in greater detail in Chapter 4. Corrosion was suspected for the delaminations on Pier 5 and the cracking on the remaining piers. Half-cell potential testing was employed at selected locations to determine if ongoing corrosion was occurring. This testing found that substantial areas of Piers 5 and 8 were undergoing significant corrosion (half-cell potential < -350 mV), and many areas had low readings (half-cell potential between -200mV and

-350mV) that could be indicative of ongoing corrosion. Pier 6 was generally free of areas with low half-cell potentials and Pier 9 did not have half-cell potential testing performed as it was not part of the initial investigation.

A total of 138 powder samples were extracted from various locations in the 5 piers. The chloride content was analyzed in 4×1 inch (25 mm) intervals between 0.5 inch (13 mm) deep and 4.5 inches (114 mm) deep for Piers 5, 7, 8, and 9. Chloride content was analyzed in 3×1 inch (25 mm) intervals between 1 inch (25 mm) deep and 4 inches (100 mm) deep for Pier 6. Pier 5 generally had chloride contents over 0.3 percent (considered the threshold of corrosion initiation for this report) at the surface, three samples higher than 0.3 percent between 1.5 inches (38 mm) and 2.5 inches (64 mm), and 1 location showing chloride concentrations of 0.49 percent as deep as 4.5 inches (114 mm), deep enough to reach the reinforcement. Chloride testing on the other piers showed generally low chloride concentrations at all depths tested. Due to the low chloride concentrations found at the surface, it was not expected these piers were exposed to exterior sources of chlorides. The background chloride concentration of the concrete, taken from the deepest samples, were also well below 0.3 percent.

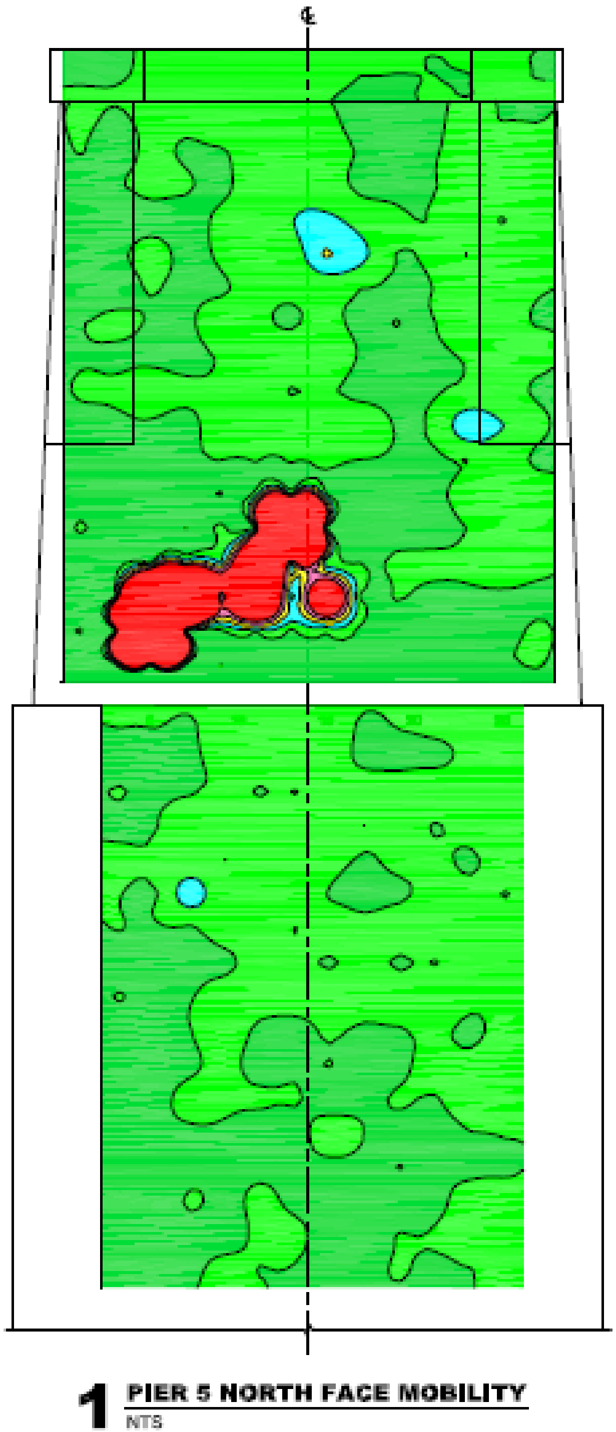

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Figure 53. Image. Delamination and cracking survey provided

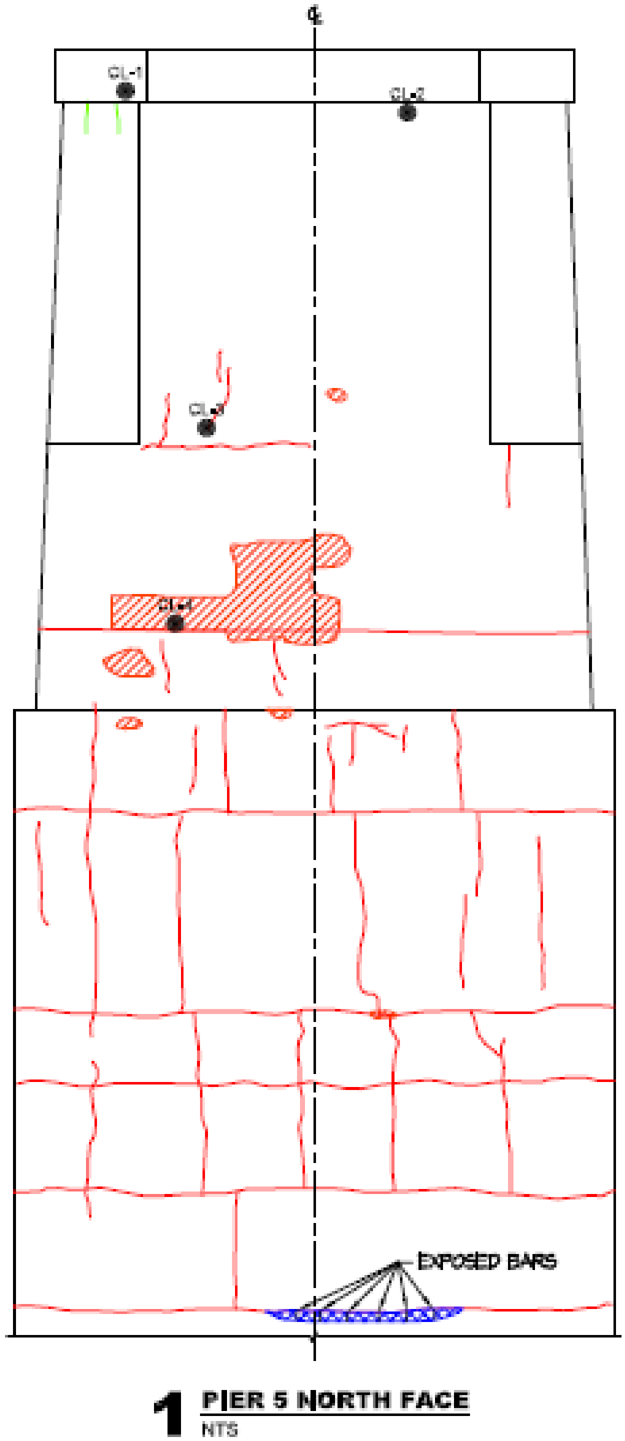

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Figure 54. Image. Mobility survey taken from IR measurements

North Torrey Pines Bridge, Del Mar, CA

Testing and Analysis Performed

Fifty four sample cores were taken from the bridge faces for carbonation and chloride testing (Johnson et al. 2015a). These cores allowed the depth of carbonation to be measured, and 270 chloride measurement samples were taken from various depths of each core. The cover depth measurements described earlier were combined with the chloride and carbonation measurements to perform a statistical analysis of the expected level of deterioration.

Cover Depth

From the measured concrete cover depths, a statistical profile of the cumulative cover depth to horizontal and vertical rebar was created. The statistical profile was created using the average and standard deviation of cover depth from over a hundred cover depth measurements.

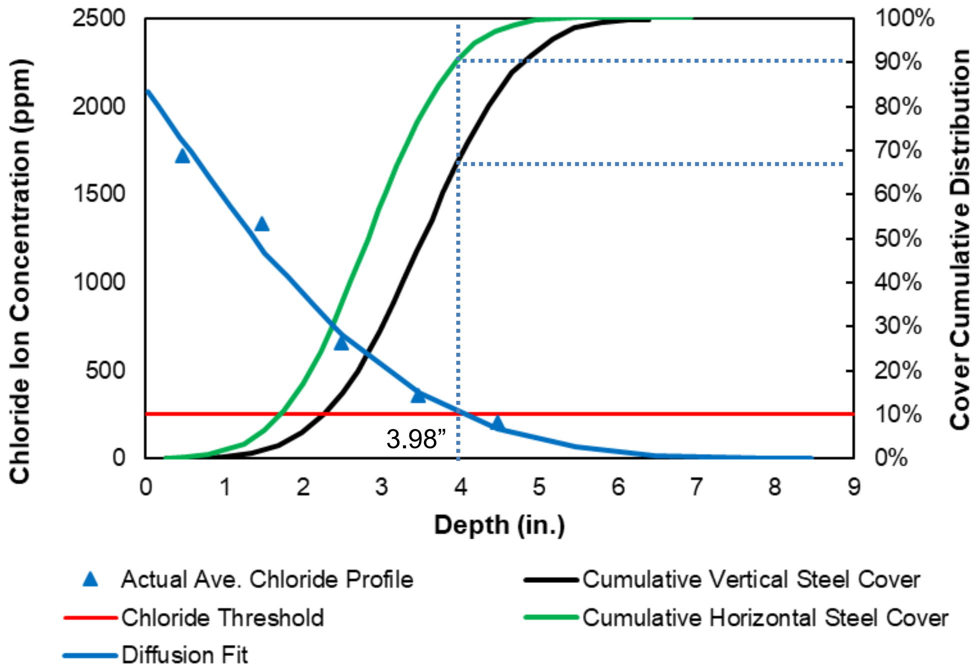
Carbonation

The minimum and maximum carbonation depth for each sample were recorded, and the average carbonation depth was found to be the average of these two numbers. The statistical distribution of the average carbonation depth was found, and a density function of carbonation depth was created. This depth was compared to the cumulative distribution function (CDF) of the reinforcement steel depth to determine that approximately 5 percent of the steel was exposed to carbonated concrete. The estimated additional carbonation over the next 50 years, was 1.86 inches(47.2 mm), and the resulting shift in distribution of cover depth implies 13 percent of the reinforcing steel would be exposed to carbonated concrete.

Chlorides

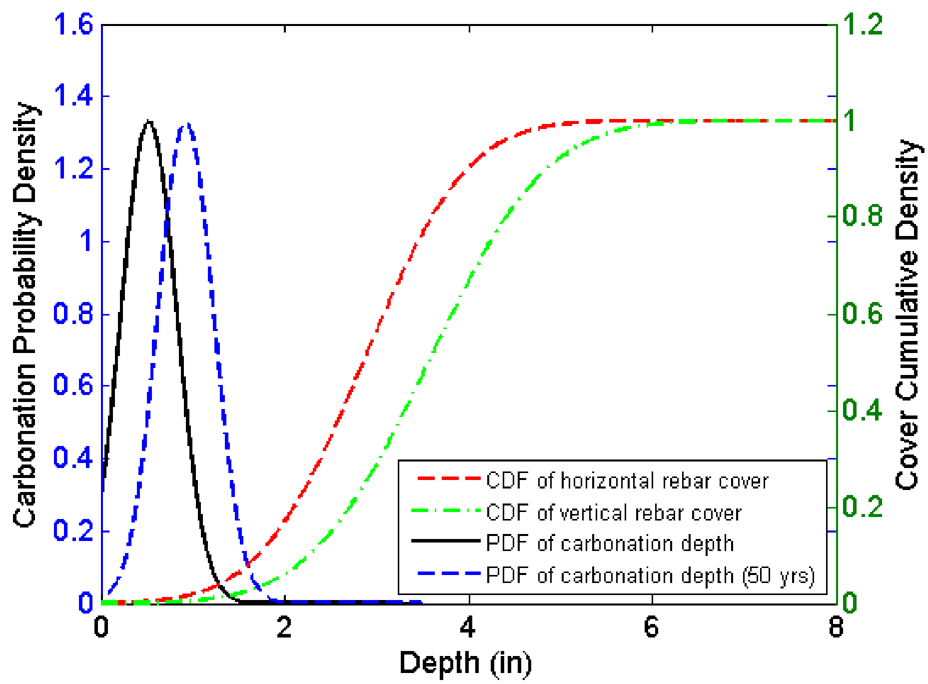
For each of the 54 cores extracted, the surface chloride concentrations and apparent diffusion coefficients were calculated from the chloride profile. The north faces of the structure were found to experience a statistically significant higher amount of chlorides than the other faces, although significant chloride ingress was observed on all faces. The north face was also the ocean facing portion. The surface chloride concentrations for north face concrete were the approximate value expected for elements exposed to seawater, even though the only exposure was atmospheric. The chloride profiles showed that concrete surrounding 97 percent of north face horizontal bars and 68 percent of vertical bars had already exceeded the threshold levels for reinforcement corrosion. The values for all horizontal and vertical reinforcement were 90 percent and 66 percent, respectively, as shown in figure 55.

The 54 samples were also analyzed for carbonation penetration using petrographic analysis. The maximum and minimum carbonation for each sample recorded. It was found that carbonation had reached approximately 5 percent of the horizontal reinforcement and was expected to reach 13 percent of the horizontal reinforcement and 2 percent of the vertical reinforcement (figure 56).



Original Photo: © CONCORR Inc. (see Acknowledgments section)

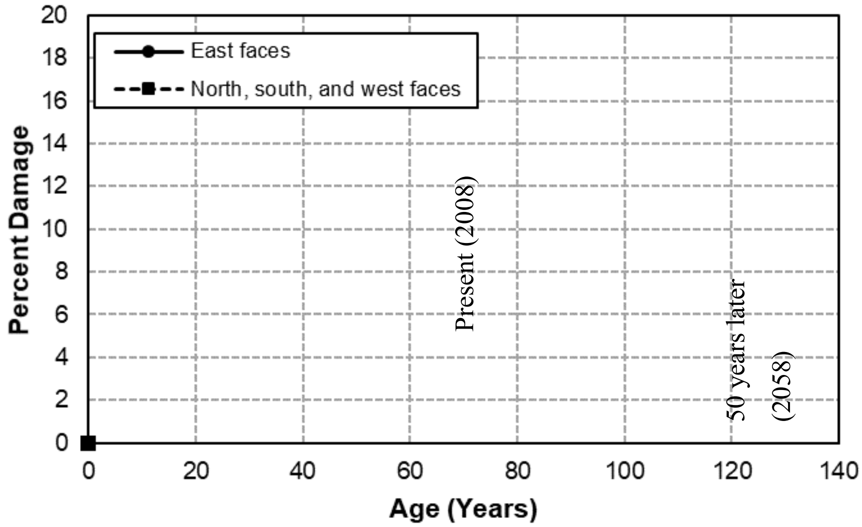
Figure 55. Graph. Chloride ion depth versus reinforcement depth



Original Photo: © CONCORR Inc. (see Acknowledgments section)

Figure 56. Graph. Carbonation depth vs reinforcing steel depth

Measurements were taken of the present damage in the concrete to perform service life modeling, following the work described by Sohanghpurwala (2006). The cumulative damage present on the piers was calculated with respect to concrete age, as shown in figure 57.



Original Photo: © CONCORR Inc. (see Acknowledgments section)

Figure 57. Graph. Percent damage on concrete elements with age

Georgia Street Bridge, San Diego, CA

Substantial cracking and deterioration had long been noted at this prior to considering reuse options. Prior testing on approach earth retention walls had shown that the concrete was of very poor quality, with substantial carbonation and a high level of chlorides recorded on the surface of the north wall, but levels below 0.3 percent were found at the south wall. Shotcrete had been used multiple times in the past to repair spalling sections of concrete. Additional testing was performed as part of the reuse evaluation program.

Additional testing was undertaken as part of the reuse evaluation program. Cores taken from the abutments showed chloride levels of 0.49 percent and 0.63 percent for surface concrete (first 0.5 inch, 12.7 mm) in the north and south abutment, respectively. Carbonation depths for the north abutment ranged from 4.5 inches (114 mm) to 5.3 inches (135 mm) and were greater than 4.8 inches (122 mm) for the south abutment. The arches showed 1.3 inches (33 mm) to 2.9 inches (74 mm) of carbonation penetration, a maximum chloride concentration of 0.3 percent on the surface, and a maximum chloride concentration of 0.08 percent at rebar depth. The deadman anchors and thrust blocks were not investigated as part of this evaluation. These elements were located permanently beneath the ground surface and were not directly exposed to air or chlorides. The anchors were reused as is, and their service life was adequate for reuse.

The final decision on reuse encased the abutments with a 9 inches (229 mm) of new concrete supported by soil nails installed through the existing wall. The old abutment walls were abandoned in place, without relying on them for capacity or any remaining life. The carbonation of the existing concrete would be largely stopped by this procedure due to the encasement, although this was not important as the new concrete and reinforcement mat could resist the lateral earth pressure and seismic loading.

The rib arches were also reused, but the cover concrete was removed and replaced with new concrete. The exterior concrete was removed with hydrodemolition to protect the interior core concrete. This procedure removed existing carbonation and chloride intrusion, and the new concrete layer was designed to resist carbonation for the expected 50-year design life of the reused bridge. The spandrels were completely removed and replaced, so their new service was governed by the quality of the new concrete.

Haynesville Bridge, Haynesville, ME

Both abutments and both internal piers of the Haynesville Bridge rested on groups of timber piles. The timber piles underlying the central piers were permanently submerged and are not considered to be susceptible to deterioration. The timber piles underlying the abutments are above the river level at the time of testing and are believed to be generally above the groundwater table, although the groundwater table fluctuates seasonally.

Excavation was performed at each abutment to conduct PIT testing and static load testing for a single pile at each abutment. For the sake of durability, the major takeaways were that no damage was observed at either pile (insects or rot), and the presence of creosote was noted during cutting of the pile for installation of the test equipment. Due to the lack of observed deterioration and the observation of intact preservative, the piles were deemed to have sufficient remaining service life.

NJ Route 72 Bay Bridges, Ocean County, NJ

The timber piles at the Manahawkin Bay trestle bridges had already undergone significant deterioration and would have limited service life without remedial actions. Some piles had already experienced enough section loss to threaten their stability, effectively meaning these piles were at the end of their usable service life. Pile jacketing repairs were applied to all piles exhibiting deterioration to prevent further decay of these piles and restore stability to piles with significant section loss. The jacketing repairs are expected to extend the service life of the repair piles indefinitely. Non-repaired piles can experience ongoing degradation that may require future repairs to extend their service life. Considering the high variability in service life observed from the existing piles, the remaining service life of the unrepaired piles was not explicitly calculated.

U.S. Route 1 Viaduct, Bath, ME

Shallow test pits were performed at 5 of the 19 piers and 1 of the 2 abutments to expose a portion of the existing pile caps and a portion of the steel piling. The excavation attempted at the abutment was unstable due to the rate that water was entering the excavation, preventing observation of the piles. One or two piles were examined on each uncovered foundation. No corrosion was observed, although some minor pitting and chipping was observed on two of the piles exposed. Measurements were made on the exposed steel sections that indicated the thickness of the flanges were equal or slightly thicker than the design criteria for the HP10x42 piles specified in the original bridge drawings. The test pits identified water intrusion near the bottom of the pile caps, indicating the steel piles were likely permanently submerged or in very wet soil just above the water table. Although no corrosion was identified, an additional 1/16 inches of sacrificial thickness was subtracted from the cross-sectional are of the pile to account for the potential of unseen corrosion.

CHAPTER 6. CAPACITY ASSESSMENT

INTRODUCTION

The chapter provides an overview of the procedures that can be employed to ensure that adequate capacity is available for the reused foundation to comply with modern codes. Foundations being considered for reuse are often decades old and were likely subjected to older design codes and construction quality standards. Documentation on these foundations may be incomplete or may contain unreliable information (pile capacity given with no driving logs or testing, incomplete details about reinforcement, incomplete test results, etc.). Very often, design plans exist, but as-built drawings, inspection records, pile driving logs, subsurface information, pile test records, and other QA/QC data may be unavailable or indecipherable. Engineers involved in the decision to reuse a bridge foundation are tasked with piecing together this incomplete data with additional subsurface investigation to determine the available capacity of the foundation. The capacity assessment builds on the findings from the integrity and durability assessments since both can directly affect the capacity.

The overall goal of capacity assessment is to prove that a desired level of capacity exists within the context of current LRFD and State DOT guidelines. The capacity assessment covers scenarios ranging from verifying original design capacity, determining LRFD capacity for a foundation originally designed using ASD or LFD, determining if an increased nominal capacity is available (if there is reserve capacity), or determining if capacity has been reduced by deterioration or damage. Possible strengthening and repair alternatives, discussed in the next chapter, can be considered during this assessment. The capacity assessment would then be useful for determining the extent of strengthening required, if necessary. Important questions that are considered part of the capacity evaluation include:

- How have codes changed since original design?
- What are the new loads on the foundation?
- What are the material properties and capacity of structural elements?
- How has deterioration affected capacity?
- What is the geotechnical capacity of the foundation?
- How will capacity be affected by projected changes? (from durability assessment)

LOADING ON REUSED FOUNDATION

One of the first steps in the evaluation of the existing foundation capacity is the determination of the future loading on the foundation. The future loading may be the same as the design loading for the original foundation or may change because of newer design provisions. Such design changes may include increased dead loads and live loads because of widening, new type of superstructures, loading from extreme hazards, or the use of lightweight materials to reduce the dead weight of the new superstructure. Loads on the foundation could also increase because of new guidelines on extreme hazards, such as vehicular impacts. Table 29 below shows possible causes of additional or reduced loads on foundations during reuse design.

Table 29. Possible causes of additional or reduced loads during reuse planning

Possible Causes of Additional Loads	Possible Causes of Reduced Loads
Wider/heavier superstructures	Lighter Superstructure (lightweight concrete, more efficient design)
Reduction in number of Piers (increased span length)	Increased number of piers (decreased span length)
Higher wind loading due to new analysis or superstructure cross section	Better wind profile, changes to later behavior
Higher seismic loads	Reduced superstructure mass resulting in lower seismic forces
Higher hydraulic/scour loads due to structural changes, new design floods/codes, or improved analysis	Scour countermeasures, new structural systems to take hydraulic loads
New design collision/impact loads	Installation of fenders, dolphins, or pier protection
Increased number of lanes	Reduced number of lanes
Heavier vehicles	Reduced soil pressures (better analysis, replacement of backfill with lightweight fill such a geofoam, changes to soil geometry)

Superstructure Alternatives

Identifying multiple superstructure design options for consideration in the early stages of reuse planning and decision making is often beneficial. Many reuse projects may involve widening (adding lanes to the superstructure to handle greater traffic volumes). Typically, this will increase the dead load on the foundation, although design changes like using a more efficient structure, lightweight concrete, or adding piers and shortening the span length can reduce the dead load demands on the foundation. The addition of traffic lanes, along with potentially heavier truck loads, will increase the live load on a bridge. Heavier decks will generally increase the seismic loads on the substructure. Design changes to the profile of the bridge deck may increase or decrease the wind loading transferred to the substructure.

By itself, the most economical superstructure design may not be the most compatible with the foundation. Design choices like the use of lightweight concrete may add to the initial superstructure cost, but may make foundation reuse feasible, or reduce the amount of strengthening required. An example of this is the U.S. Route 4 Bridge over Ottauquechee River in Vermont (FIU 2017), where the existing substructures were encased and strengthened, but lightweight concrete was used in the bridge deck to reduce loading. The use of a lower weight alternative may also allow the engineer to reduce the amount of substructure/foundation investigation necessary, although the cost of investigation is often insignificant when compared to total construction costs.

Modeling

Various methods of modeling are commonly employed to determine the forces, moments, and stresses experienced by substructures. A list of available methodologies that can estimate the loading on substructures is provided in table 30. The most simplistic of approaches is an approximate analysis, where the forces applied by the superstructure are assumed to be known and the foundation is analyzed as fixed. The fixity can be a distributed fixity underneath the footing or point fixities at the connection of a pile cap to the supporting piles. This type of analysis does not typically include the contributions of side soil to lateral resistance, and as such can overestimate the structural demands.

Finite Element Analysis (FEA) is commonly used to provide more accurate estimation of complex geometry and loading. Various FEA packages are available with many of the same capabilities, although many DOTs have a preferred package. A wide range of complexity is available, from linear frame elements supported by fixed conditions to non-linear 3D models where piers and abutments are modeled with solid elements. Linear or nonlinear springs (P-Y, T-Z, T-θ, and Q-Z) can be applied to FEA models or used in finite difference programs to predict the response of foundation elements to loading. Complex methods for modeling soil behavior include full 3D solid elements with soil properties and springs representing soil behavior. When modeling the lateral resistance provided by soil, it is important to consider the potential for scour at the bridge, which can erode soil that provides lateral resistance in the non-scoured condition.

Table 30. Modeling Methodologies for determining loading on substructure

Modeling Methodology	Advantages	Disadvantages	Notes
Approximate analysis	Simple, based on basic engineering knowledge, finds forces/moments in the substructure elements	Cannot account for non-linear soil contributions, or complex geometry	Generally good as a first estimate of loading on the structure and soil. Can be difficult to account for behavior of soil.
Finite element analysis	Allows input of entire system, including soil and wind, can be non-linear, allows input of structure to help determine actual forces going into substructure elements	Can be computationally expensive for large bridge, may require many hours of input, requires knowledgeable staff to create model, requires knowledge or estimation of many material properties	FEA models can be of entire bridge system, or individual substructure elements. Can include non-linearities if needed, although at increased model complexity.
P-y curves (and T-Z, T-Θ, Q-Z)	Simplistically includes soil behavior, can be applied to FEA models, reduces computational cost while approximating soil behavior	Requires experience, difficult to apply to large diameter piers in deep soil	Simplifies input of soil behavior through non-linear springs. Linear springs can be used if they appropriately approximate the soil behavior in the load case being considered.
Wind tunnel testing	Accurate estimation of wind forces acting on superstructure and substructure	Expensive, time consuming	Allows determination of wind forces transferred to substructure using scale models. Generally, only used for signature bridge with unique profiles and complicated wind analysis.

P-y curves are commonly employed in geotechnical evaluation and in programs such as LPILE and FB-Pier. These curves typically represent the relationship between lateral movement and soil reaction. Similarly, T-Z curves represent axial displacement – skin friction behavior, Q-Z curves represent axial displacement – end bearing behavior, and T-Θ curves represent twisting – torsional resistance. By nature, these curves are non-linear, but they can be further approximated as linear springs specific to the ultimate condition of each load case. These curves can be applied in a finite difference scheme for individual piles, as done in LPILE, or they can be applied directly to FEA models.

Wind tunnel testing is typically reserved for signature bridges that require highly specific analysis of the wind loads. These models use a scaled model of the entire bridge, allowing for the estimation of the total wind load applied to the structure and each of its substructures.

Unload Monitoring

Unload monitoring can be performed on reused foundations through implementation of an "unload test" (Bell et al. 2013). During an unload test, the rebound of the foundation is monitored while the original superstructure is removed. By obtaining these measurements, the settlements of the foundation with new superstructure loads can be predicted. Unload monitoring is very effective at predicting the immediate settlement of the foundation when the new superstructure load is applied, especially the new superstructure dead load is equal or lower than the original dead load. This method may not fully predict the future settlements after time and exposure to live loads, as these may be higher than was applied during the previous service life.

Deflection Criteria

Samtani and Kulicki (2016) proposed additions to the LRFD Bridge Design Specifications (AASHTO 2014) that include deflection criteria. The deflection criteria were calibrated using a reliability analysis that assumes a non-probabilistic tolerable settlement, along with probabilistic potential settlements. The results from this report can be considered in conjunction with the findings from unload monitoring to better estimate potential settlements when placing new loads on the existing foundation.

GEO/HYDRAULIC HAZARDS

Geo/hydraulic hazards can induce loading on bridge foundations that may not have been present during the initial service life of the bridge. Often, an older foundation under consideration for reuse was designed to different design codes that did not evaluate these hazards as stringently as modern codes do. Even though an existing foundation has a history of adequate performance, it may not necessarily have been subjected to these loads in the past. Long-term changes like rising sea levels or precipitation changes may impact the return period of certain flooding events, requiring increased scrutiny of the design flood and its impacts. Two major hazards that can induce loading on bridge foundations include scour and seismic activity.

Scour

The LRFD Bridge Design Specifications (AASHTO 2014) states in Article 3.7.5: "The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at strength and service limit states. The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit states." The "design" flood used to evaluate foundations for strength and service states represents a 100-year flood, or the maximum flood event with an annual probability of exceedance of 1 percent. The bridge is expected to remain operable with full traffic loads after sustaining this amount of scour. The "check" flood used in the extreme event cases represents the 500-year flood, or an event with a 0.2 percent chance of annual exceedance. A review of analysis techniques that can be used to evaluate the scour and water loading associated with these floods can be found in HEC-18 (Arneson et al. 2012).

There are three main sources of scour that contribute to the total potential scour at a bridge pier or abutment: long-term changes to the riverbed, contraction scour, and local scour. Long-term changes consist of aggradation or degradation, which refer to the deposition or removal of sediment from long stretches of the river bed, respectively (Arneson et al. 2012). These processes are not generally influenced by the presence of the bridge but are caused by long-term hydrological conditions that influence sediment transport. These stream bed changes are not typically considered to be part of flooding events, although these changes can exacerbate the impacts of flood-induced scour. Contraction scour occurs across the entire width of the stream bed near a bridge because of increased water velocity due to a reduction in channel width. This type of scour can be especially prevalent when abutments are much narrower than the normal or flooded stream bed width. Contraction scour can occur due to normal stream flows, cyclic stream flows (such as tidal fluctuations), or from floods that cause elevated stream velocities near the bridge. The third, and often most impactful type of scour is local scour, which occurs immediately adjacent to foundation elements located within a moving stream. Local scour results from vortices generated when normal stream flow is obstructed. These vortices can have high velocities which cause sediment to become dislodged and transported away from the foundation. Local scour holes can become filled in with debris and loose soil that do not provide reliable geotechnical resistance and is typically neglected in calculations. In addition, the presence of river borne debris can impact local scour, as discussed in NCHRP 653 (Lagasse et al. 2010).

Seismic Loading

Seismic demand (D) on bridge components is typically evaluated in relation to their capacity (C) using the ratio of C/D. A C/D ratio of 1.0 or greater implies that the bridge has sufficient capacity to resist the ground motions under consideration. Two main demands are considered during seismic analysis: force/moment demand and displacement demand. Force/moment demands refer to the actual loading that a component will experience during the design seismic event. Displacement demands, typically calculated from a pushover analysis, allow non-linear behavior (hinging, yielding, engineered hinges) of the bridge to be considered during design. The calculated displacement demand is used as a measure of the amount of ductility provided to a bridge component that is expected to undergo significant inelastic behavior during a seismic event. Since earthquake can induce such high loads, it is often beneficial to allow the formation of a plastic hinge, or inelastic behavior where a component can continue to deform without taking much additional load. This behavior allows loading to shed to other components without there being complete failure of the hinging section, although the hinging action can cause significant damage to that component. When a component lacks sufficient displacement capacity, it can catastrophically fail and lead to collapse of the bridge. Pushover analysis is generally appropriate for analyzing the maximum considered event (MCE), where life safety is protected but some damage may be allowable. If the bridge being considered for foundation reuse is a critical bridge, then it may be important to consider the damage level during an MCE so that the traffic can be restored following an MCE after inspection. Various techniques are available to evaluate seismic demands, and the appropriate approach depends on the bridge details, the magnitude of expected seismic events, and the importance of the bridge. Detailed seismic analysis and design of the bridge can be carried out by following the provisions in the LRFD Bridge Design Specifications (AASHTO 2014), AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2011) with interim revisions or local seismic design specifications, such as New York City Department of Transportation Seismic Design Guidelines for Bridges in Downstate Region (NYCDOT 2016).

COLLISIONS AND OTHER HAZARDS

Wind

Wind loading acts on bridge superstructure and piers to impact lateral and moment loading on foundation elements. Two level of wind loading are considered: operational 55-mph winds that coincide with normal traffic (and evaluated in Strength V, and Service I load cases), and extreme wind events with wind speed determined by location and exposure (considered in the Strength III and Service IV load cases.) When a new superstructure is placed on top of an existing foundation, the new superstructure may have a different wind profile that alters the lateral loads induced on the foundations by wind. In addition, maximum wind speeds (and their return periods) may be impacted by development near the bridge or changing climatic conditions. Depending on the type of new superstructure, analysis and design of the bridge can be carried out by following the provisions in the LRFD Bridge Design Specifications (AASHTO 2014) or other design specifications adopted for a particular bridge site.

Impacts/Collisions

Impacts can occur to bridge foundations in the form of vehicle collisions or marine vessel collisions. Damage to existing foundations from previous collisions is identified and assessed during the integrity assessment, as discussed in Chapter 4. The design standards surrounding impacts have become much more thorough in recent years, and as such many older foundations may not have been designed for modern impact loads. Retrofitting these older foundations usually consists of either encasing the existing foundation in enough concrete to provide resistance through shear mass or to provide protection systems, such as dolphins or fenders. The of dolphins in scourable piers may impact the scour analysis (Zevenbergen et al. 2012). These systems are often constructed from driven timber/steel piles, concrete walls, or other massive structures than can deflect or soften the impact of a large vehicle. Bridge substructures vulnerable to marine vessel collision can be designed by the provisions in the LRFD Bridge Design Specifications (AASHTO 2014). Bridge piers vulnerable to truck impacts can be analyzed and designed by following the performance-based approach available in Agrawal et al. (2018).

Fires

Fires most often impact bridges when a vehicle or structure burns underneath a bridge overpass. The heat from this fire can cause yielding and weakening of steel and cause cracking of reinforced concrete. PCA (1994) discusses the analysis and repair of concrete members that have been previously exposed to fire, as discussed in Chapter 4. NCHRP 12-85 (Wright et al. 2013) discuss how to analyze the risks associated with fires at bridges, how to predict structural response to fires, and post-fire evaluation of bridge components. Computational fluid dynamicsbased approaches for analyzing the behavior of bridges during fire are available in Gong and Agrawal (2015; 2016).

Ice and Debris Flows

Flows of ice and other river borne debris can impact bridge piers and abutments and provide substantial additional lateral loading to foundation elements. NCHRP 445 (Parola et al. 2000) investigated the impact of debris flows on bridge piers and provided draft specifications for deign against debris flows. Zevenbergen et al. (2012) has published an FHWA guideline on the hydraulic design of safe bridges, including design against flows of ice and debris. The LRFD Bridge Design Specifications (AASHTO 2014) currently include specifications on the loading that can be introduced by debris and ice flows on bridge piers.

Blasts

Loading from blasts or explosions is a relatively new consideration for designers of civil infrastructure. Blast loading presents a difficult challenge due to the high forces and unpredictability of blast loading. Recently, the FHWA has published the Bridge Design Security Manual (Davis et al. 2017), which covers the types of blasts expected at bridge sites, the how materials react to this loading, and guidance on how to design for these effects. NCHRP Report 645 (Williamson et al. 2010) provides design and detailing guidelines for blast-resistant highway bridges.

CHANGES TO CODE REQUIREMENTS

Many older bridges have substructures originally constructed following ASD or LFD standards. In NCHRP Synthesis 505 (Boeckmann and Loehr 2017), 39 U.S. state agencies provided a response to the question "Under what design guidance or standards has your agency evaluated foundations to be reused?" Table 6-1 shows response of the survey. Most respondents (79 percent) stated that AASHTO specifications from the time of reuse had been applied to reuse cases, 38 percent of respondents replied that design standards from the original construction were followed, and 28 percent of respondents replied that the State design provisions had been followed, as shown in table 31. Several agencies provided multiple responses.

Table 31. Design standards applied to reused foundations

Response	Number	Percent
AASHTO specifications from date of foundation reuse	31	79%
AASHTO specifications from date of original foundation construction	15	38%
State-specific provisions	11	28%

Since reused foundations are expected to be in service for another full lifespan, it is often desirable to follow current codes for bridge foundations, rather than those at the time of original construction. This will generally consist of an approach consistent with LRFD and/or local provisions. Many older bridge foundations that are candidates for reuse have been designed using working stress type designs (e.g., ASD, LFD). This switch from old to new codes will require reevaluation of both the loading and capacities of the foundation for reuse. In practice, there have been cases observed where the original working stress codes were reapplied to simplify the redesign of the foundation. Existing design plans from the original time of construction will often include the capacity of the element, as determined in accordance with the codes applicable during original construction. For substructure constructed based on ASD or working-stress design provisions, the capacity of the original foundation may not necessarily correlate to the capacity based on LRFD specifications.

CALCULATION OF STRUCTURAL CAPACITY

Pier Columns and Wall Piers

The critical load cases for pier columns and walls are often those with significant lateral loading that induces moments in the column or wall. Load cases in AASHTO (2014) with significant lateral loading include extreme event cases with seismic loading, vessel/traffic collision, or ice loading; and Strength III, Strength V, Service I, or Service IV load cases that include wind loading on the substructure and superstructure. Elements that are under-reinforced or have significant rebar corrosion also may not have sufficient resistance to tensile cracking. When evaluated at Service Limit states, it may be necessary to prevent tensile stresses from occurring, rather than using the modulus of rupture, as given in Section 5.4.2.6 of AASHTO (2014). Strength Limit states are still generally evaluated considering the modulus of rupture, unless there is significant cracking observed in the tensile portion of concrete. Various construction sequence issues can arise for piers during reuse. Additional loads that may need to be considered at this time include: construction loads related to traffic rerouting, bridge sliding, deconstruction activities, additional ground movement, and construction equipment.

Abutments

Abutments and wingwalls for reused foundations can be subjected to vertical settlement, horizontal movement, rotations, and slope stability problems. Any bowing or excessive movement of the wall is an indication that the wall is unable to support the soil and surcharge loads in the “at-rest” state. As the wall continues to move, the soil pressures acting horizontally on the wall will transition to the active state, where the coefficient of lateral earth pressure is lower. This transition reduces the horizontal load on the wall but requires a certain amount of movement to mobilize, given by table C3.11.1-1 of the LRFD Bridge Design Specifications (AASHTO 2014). Similarly, soil in front of the abutment that is restraining wall movement will begin in the at-rest state and transition to its passive state as the wall moves. In the passive state, a wedge of soil is providing resistance to wall movement and the earth pressure coefficient increases up to several times over the at-rest coefficient. Since the passive state increases resistance to wall movement and the active state reduces the earth pressure load driving movement, wall movement generally increases the factor of safety of the wall.

The at-rest lateral earth pressure coefficient represents the maximum earth pressure than will be generated by soil being retained. The LRFD code allows for use of a factor of 1.35 for at-rest soil pressure calculations. Since the active lateral earth pressure coefficient is the minimum theoretical value for a wedge of soil, a greater factor of 1.5 is placed on the loading to compensate for the greater potential of non-conservatism if the active condition is not fully reached. Still, the reduction of active soil coefficient achieved by using the active earth pressure generally outweighs the additional factor of safety. For instance, assuming a scenario with normally consolidated soil with $\phi = 30°$ and flat backfill, $K_0 = 1\text{-}\sin\phi$ is equal to 0.5, the Rankine active coefficient is 0.33. The factored K_0 would be 0.675 and the factored K_a would be 0.5. As the friction angle increases, the benefit of using K_a or K_p over K_0 generally increases, depending on the equations used.

Estimating the currently observed amount of movement at an abutment allows inference to the behavior of the foundation as well as the loading on it from lateral earth pressure. The observed movement can be interpolated between the values for K_a and K_0 to estimate the actual pressure being applied to the abutment or retaining wall.

Pier Caps

Pier caps often experience significant shear and flexural demands that govern their design. For reinforced concrete pier caps, this requires evaluation of both transverse (shear) and longitudinal (flexural) reinforcement prior to capacity estimation. Frequently, the shear reinforcement sustains the most damage due to corrosion as it outside of flexural reinforcement.

Pier caps are often analyzed using a strut and tie model, as discussed in Section 5.6.3 of AASHTO (2014). This method analyzes the section treating flexural and shear reinforcement as ties in tension, with concrete acting in compressive struts. The quality and compressive strength of concrete is most important in the strut regions, as these transmit compressive forces.

Piles and Pile Bents

Steel pile bents often will have experienced at least some minor corrosion since their installation, which may not be uniform. This corrosion can be directly measured and accounted for during the integrity analysis if the element is exposed and/or accessible. Corroded areas will be unable to provide any strength, and additional deterioration may be expected, requiring an amount of sacrificial thickness. The amount of sacrificial thickness required is governed by design life, environment, and by initial condition of the foundation.

When evaluating timber piles, the wood strength value follows from the integrity evaluation, which considers both typical values and potential testing. Elements that have lost section due to impacts, decay, marine borers or other issues are typically analyzed with reduced cross sections. Service Limit based load rating procedures considering section loss have been developed (Andrawes and Caiza 2012), by reducing the amount of timber available for resistance to loads. Andrawes et al. (2017) have provided a method for load rating of deteriorated timber piles repaired by wrapping with FRP. The Strength Limit state capacity of timber piles can be determined using the cross section of element, or an assumed reduction of the pile area or moment of inertia.

DRIVEN PILE CAPACITY

Geotechnical Data

Obtaining the original geotechnical data (e.g. soil borings, SPT data, lab testing) is important to analyzing driven pile foundations. Not only does this data provide information on the soil system, it provides an insight to the original design intent behind the in-situ piles. Regardless of the quality of the existing data, it is considered good practice to obtain at least some additional geotechnical data on a foundation prior to reuse. This additional data can be as simple as borings with SPT blow counts to confirm the existing data. When available data is less extensive, a greater amount of soil investigation might be needed to be undertaken for there to be sufficient confidence in the capacity of driven piles.

Previous Foundation Testing

Pile test data is the single most important piece of information available for determining the capacity of pile foundations. Considering the age of the typical reused foundation, it is unlikely that dynamic testing was performed at the time of installation. Any test data is likely to be static load test data. Ideally, the test data includes at least one pile loaded to failure. Such test data provides a direct measurement of the pile capacity at the time of the test. Since the test, the capacity may have changed if portions of the pile have experienced scour, downdrag, or additional set up. As these uncertainties exist even for a newly installed pile, it is commonplace to account for the loss in side resistance from scour or downdrag when determining the pile's ultimate capacity. If additional set-up has occurred, the piles may have more capacity than the original test data would suggest, but it is not recommended that this capacity be relied upon without new test data that measures that effect. If the original piles were not loaded to failure during the testing, it is possible that additional pile capacity beyond the original design may be available, since the original design is likely to be based on the tested capacity (which will be lesser than the capacity of the pile at failure).

Due to their high capacity, it is less practical to load test drilled shafts at the time of installation. The design of these elements is more reliant on static capacity calculations and verification during installation. When static compression test data is available, the LRFD Bridge Design Specification (AASHTO 2014) allows a resistance factor of 0.7 to be used. Sites with highly variable soils or where there is not high confidence on the consistency of production shafts may use a lower resistance factor, subject to engineering judgement.

Pile Set-up

The capacity of an existing foundation may change from the originally tested capacity because of factors such as scour, downdrag set-up and settlement. Scour removes soil previously available for resistance, downdrag removes soil and impart a vertical load, and setup of the side soil increases available side friction. In addition, settlement during the initial service life may mobilize increased resistance and inelastic settlement behavior, which can limit the potential for future settlement.

Impact of Scour and Downdrag on Pile Capacity

Driven piles often support foundations that are susceptible to scour or downdrag. Scour susceptible piles are found in bodies of water or in flood plains and can have soil along the sides of the piles eroded away during flood loading. The current LRFD Bridge Design Specifications (AASHTO 2014) require that the contributions of soil that will be scoured during a 100-year flood event not be included for strength and service limit checks. When pile capacity is determined from driving equations or testing, the total tested capacity includes resistance provided by layers that would be scoured during a 100-year flood event. In practice, these contributions are estimated using design equations and subtracted from the tested capacity to determine the capacity during a 100-year scour event. When reusing an already scoured foundation, it may be possible to test the piles in the scoured condition to directly measure the scoured capacity. This capacity can be further altered if additional scour continues to occur. Foundations that can be undermined during scour (material is lost beneath the pile tips) are typically referred to as "scour critical" and can present a major hazard during a hypothetical future flood event. Scour critical foundations are not typically candidates for reuse without mitigatory actions to prevent or reduce potential scour. There is an important distinction between the effects of local scour and global scour (such as contraction scour or stream degradation). Local scour will primarily impact the lateral resistance provided by the side soil, whereas global scour will impact the lateral resistance and lower the overburden pressure at the pile tip, potentially impacting the pile's end bearing capacity as well. The various types of scour and further discussion of scour analysis is provided in Chapter 8.

Driven piles can also be found on sites with large deposits of compressible material (e.g. soft clays, liquefiable sands, organic materials, etc.) found along the sides of the piles. Like scourable material, the contributions from downdrag-susceptible material are typically subtracted from the tested capacity when estimating the capacity of the foundation after downdrag has occurred. Downdrag not only stops the consolidating soil layers from providing resistance, it also induces a load on the piles that uses up some of the pile capacity. AASHTO (2014) specifies that both phenomena be accounted for when estimating pile capacity.

Rebound Monitoring

Rebound monitoring, where the pile rebound is measured during removal of the original deck and superstructure loads, allows for the estimation of the settlement that will occur when that load is placed back on the pile. As a generality, the rebound movement will be substantially lower than the initial settlement, and reloading (with a new deck) will occur along the unloading curve. If the load is increased beyond the past loading, it will not necessarily follow the unloading curve, and the foundation will generally show less stiffness. The resulting settlement could be substantial if the pile begins to undergo plunging failure.

In-situ Capacity without Test Data

When piles or drilled shafts are reused, it can be difficult to assess the capacity in accordance with modern code requirements. This includes appropriate procedures during evaluation, testing, and load factors. Various State DOTs have prescribed procedures for acceptance testing of piles, either using dynamic, statnamic, or static testing. The LRFD Bridge Design Specifications (AASHTO 2014) provides resistance factors for use with various static capacity calculations. The resistance factors required for calculating static capacity of driven piles are available in table 10.5.5.2.3-1 of the LRFD Bridge Design Specifications (AASHTO 2014). These factors range from 0.25 to 0.50. Drilled shafts designed using static calculations have resistance factors from 0.45 to 0.6. The FHWA-modified Gates driving formula that considers end-of-drive criteria has a required resistance factor of 0.40.

In-situ capacity of foundations can be verified from "unload test" during some cases where it is possible to instrument underground foundation elements. Bell et al. (2013) have presented a unique method for measuring the loading on existing building foundations by installing fiber optic strain and temperature sensors along existing drilled shafts supporting an eight-story building in London and then monitoring during the demolition of the building. This "unload test" resulted in profiles of strain, heave, and load in the shaft after six stories had been removed, which could be used during the evaluation of the capacity of foundation elements. However, this measured information only corresponds to current loading on the foundation elements and does not gives additional information on the ultimate capacity. The "unload test" is also an opportunity to collect valuable performance data without the need for costly loading and reaction systems during foundation reuse projects.

Geotechnical Capacity of Reused Driven Piles

The geotechnical capacity of a driven pile is influenced by the soil conditions at the site and details of the installation process. Important details for driven pile capacity include EOD criteria (blow count and refusal criteria), conditions encountered during driving, and geometry (layout, batter, depth) of the piles. Furthermore, pile capacity can be impacted over time by degradation or damage. The assessment of driven pile capacity is reliant on various forms of documentation, including design drawings, as-built details, installation logs, and test data. Design drawings are a crucial component of understanding these items, and pile foundations without design drawings present additional risks and considerations that may make reuse infeasible. The remaining documentation is all highly useful in reducing the risks of driven pile reuse, but generally not necessary, since new evaluations can be performed to fill in knowledge gaps. The assessment of the geotechnical capacity of driven piles broadly fits into four distinct categories:

1. Design drawings and test data based on initial installation available
2. Design drawings for the foundation available, but no static load test data exists
3. No static load test data exists, load testing is performed on the entire foundation to confirm capacity
4. No static test data exists, but individual piles can be tested using the pile cap as a reaction block.

Category 1: Design Drawings and Original Test Data Available

Current practices for new foundations correlate static or dynamic test data to a driving procedure (pile depth, EOD blow counts), followed for the remainder of production piles. For an existing foundation with test data, this can be confirmed through driving logs, if available. As dynamic testing is relatively newer, much of test data for foundations under consideration for reuse is likely to be static load test data. The following items are important to consider for piles with design drawings and test data.

- The test data is representative of all the piles in that site condition
- The depth matches the intended design
- The piles or drilled shafts still have the originally intended capacity

Pile driving logs are a major source of quality assurance and provide important data on the capacity of the piles. The most crucial information available in pile driving logs is the as-built pile depth. Since pile driving can become obstructed and soil layer depths can vary greatly, the design depth and actual depth of driven piles can vary significantly. The as-built depth of the driven piles is an important consideration for scour analysis, capacity estimation, and liquefaction evaluation. If the as-built depths are not available, confirmation of the pile depth can be obtained using technologies like parallel seismic (PS) or induction field (IF), as discussed in Chapter 4. Pile driving records may also contain the end of driving (EOD) blow counts. Establishing that the EOD criterion is consistent between test and production piles provides confidence that the test data is representative of all piles. If pile driving logs are not available, there is greater uncertainty to the capacity of the untested piles. This can be taken into consideration by reducing the resistance factor, as suggested by Section C10.5.5.2.3 of the LRFD Bridge Design Specifications (AASHTO 2014).

Two different types of static test data may be available: data from tests performed to a predetermined load or data from tests where piles were loaded to failure. Failure can be defined using a criterion such as Davisson (1972) or a specified amount of total settlement. Static loads tests are typically performed fully to failure, indicating a known capacity at the time of testing. If piles were not loaded to failure, they may potentially have had additional capacity which was not accounted for during the original design. The effects of downdrag and/or scour may reduce the capacity below the tested capacity. Pile capacity calculations can determine the axial resistance lost to downdrag and scour, and the increased loading from downdrag. The effects of pile setup were likely to have been included in the original test data, as it has long been common practice to allow for a time gap between installation and testing. Still, setup can occur in some slow draining soils over a long period, meaning additional "reserve capacity" may be available. However, reliance on this additional capacity typically requires some form of testing.

Category 2: Design Drawings, no Load Test Data

For many existing foundations, sufficient design details about the geometry may exist without any test data. The LRFD Bridge Design Specifications (AASHTO 2014) allows for driven pile capacities to be derived from design equations, although with a smaller resistance factor than that allowed with test data. If driving logs containing EOD criteria followed during the test are available, then the nominal capacity can be assessed by wave equation analysis, the FHWA-modified Gates dynamic pile formula, and the ENR formula. The FHWA-modified Gates formula is shown in Eq. (26).

$$R_{nd} = 1.75\sqrt{eE_r}\log(10N_b) - 100 \tag{26}$$

where E_r = energy of the pile driving hammer, e = efficiency of the hammer and N_b = number of blows required to penetrate 1 inch (25 mm). A prescribed resistance factor of 0.4 with equation (12) implies that the use of Eq. (12) may result in an inefficient design. Further, all three dynamic equations cannot be used if the pile driving hammer type and energy are unknown. In those cases, EOD data is only useful for comparing production piles with test piles.

When only the geometry is known, pile capacity can be calculated using one of the static design equations listed in Table 10.5.5.2.3-1 of the LRFD Bridge Design Specifications (AASHTO 2014). The resistance factor corresponding to these methods ranges from 0.25 to 0.45. The use of Cone Penetration Test (CPT) data with the Schmertmann method allows for a resistance factor of 0.5 to be used. CPT testing can be performed as part of the soil investigation for reuse of the foundation.

Category 3: Testing for Future Loads on Foundation

Unlike new foundations, pile foundations being considered for reuse have a documented history of load-carrying ability. Not accounting for this load-carrying history ignores an important piece of information which improves confidence in the load-carrying ability of the piles, and consequently the reliability of the piles. When pile foundations exist without previous test data, use of static calculations or EOD criteria alone may lead to excessively conservative designs. Using the past loading history on these piles can allow for more capacity. Some code guidance exists on this. For example, provision 1808.2.7 of the New York City Building Code (NYCBC 2014) allows for half of the previous loading applied to a pile to be reused. For bridge foundations, a future test load, where a known load is placed on an entire pile cap, can be employed to ascertain the capacity of the foundation. Here, the future test load is calculated to be addition load, that combined with current dead of the bridge.

Testing is performed on bridge foundations by placing additional loads on the bridge deck. When evaluating an in-service foundation, this testing can be done before removing existing superstructure without requiring access to individual piles. The load-carrying capacity of other substructure elements, like walls, pile caps, columns can also be verified during this procedure. Future load testing for foundation elements is limited to situations where the existing superstructure can handle enough loading to transfer sufficient loads to the foundation. Figure 58 shows conceptual schematics of this testing for foundation elements. A suggested general procedure for future load testing for reuse is presented in this section.

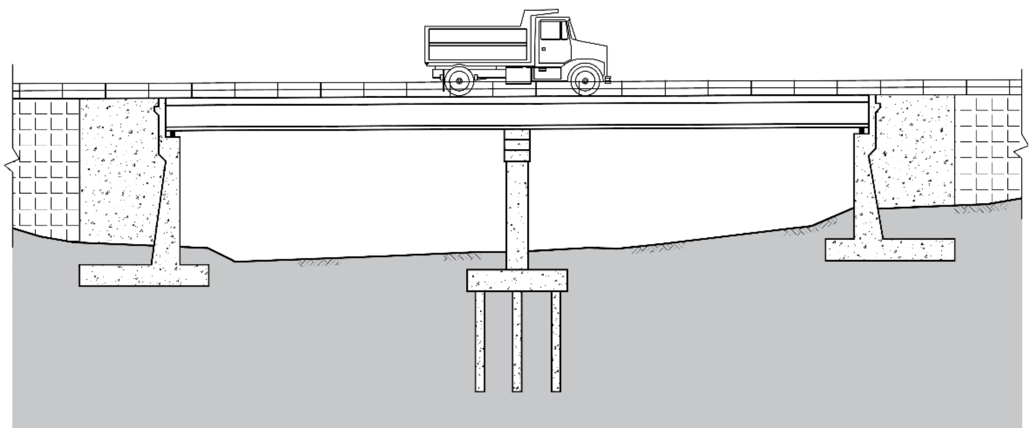


Figure 58. Illustration. Schematic of load test performed with a heavy truck

Structural Capacity and Test Loading

The first step in designing the load test is to establish a maximum safe test load for the superstructure and substructure elements to be tested. Davis et al. (2018[b]) recommend the placement of this load be controlled and engineered to induce a concentric vertical load on the foundation elements. If there is significant eccentric loading due to horizontal loads (e.g., earth pressure), the loading can be positioned so the resultant loading is nearly concentric at the expected final future test load. The maximum safe test load may be governed by the failure of any element loaded during testing. The test loads can consist of anything with mass: diagnostic load trucks, steel weights lifted into place by a crane, water barrels, etc. The use of diagnostic load trucks will involve at least some personnel on the bridge while the load is being placed onto the bridge and will require the trucks to be driven into place. Increasing the test loading incrementally allows for the response of the superstructure and movement (settlement and tilt) of the foundation to be monitored, helping to prevent overload. Loading by placing heavy weights with a crane can be done in a precise manner without any moving loads.

Future Loading

The next step of the suggested procedure is to determine the current and future loads on the foundation. The calculation for dead load is separated into contributions from elements which will remain, and elements slated for removal. The dead load contribution from elements to remain in place will automatically be included in the test. The dead load contribution from elements slated for removal will be subtracted from the dead load from proposed new superstructure and other new elements to determine the increase in dead load during the reuse phase. This increase in dead load due to the new superstructure will have some level of uncertainty that can be accounted for during the future load testing.

Downdrag, Scour, and Set-up

Soils that are prone to consolidation or may have consolidated during the original lifespan of the bridge can reduce the available pile capacity. If the soils have already consolidated and subjected to downdrag forces, then any testing program will inherently account for these changes. If the downdrag has not already occurred, these potential effects can be accounted for in the same manner that it is for new foundation design. If the material around the sides of the pile is susceptible to scour, the contributions from these layers can removed from the tested capacity. As the foundation has been in place for some time, any pile set-up which is going to occur will likely have already occurred and will be inherently included in the testing.

Differences between Traditional Testing and Future Load Testing

Traditional static load testing definitively establishes the load-carrying capacity of a single pile. Due to inherent site variability, the capacity established through testing is factored to maintain a reliability index of 2.33 for all piles installed in that site condition. Since the LRFD code is calibrated to control the reliability index of individual piles, it is suggested that the future load testing also update the reliability of individual piles. The future load testing does not directly establish the capacity of individual piles. However, as the test load is applied, the total foundation movement can be monitored, and if any single pile begins to undergo plunging failure, the load will simply shed to other piles. Therefore, it cannot be assumed that the total test load applied to the substructure is equally distributed among all piles. Rather the test will be able to establish that the total resistance of all piles is at least as large as the bridge dead load plus the future test load. In other words, the *average pile capacity* can be calculated through this testing as the sum of the test load and the existing dead load divided by the number of piles.

Failure Criterion

The proposed failure criterion for loads testing is consistent with the failure criterion typically applied to single pile testing. The most typical failure criterion used for pile testing is described by Davisson (1972), which accounts for the elastic shortening of the pile using equation (27):

$$\Delta = \frac{QL}{AE} \tag{27}$$

where L, A, and E are the length, area, and modulus of elasticity of the pile, respectively., Q is the load applied to the pile and Δ is the elastic shortening of the pile. The failure criterion, S_f is then plotted as a function of Q, using equation (28):

$$S_f = \Delta + (0.15 + 0.008b) \tag{28}$$

where Eq.(28) provides the allowable deflection in inches, Δ is found using Eq.(27), and b is the pile diameter, in inches.

This criterion is typically applied to individual pile head during normal testing, and it can be applied to the worst-case pile head (pile with most movement) during the future load test. Measurements can be taken from various portions of the pier and extrapolated to determine the movement at pile heads. By monitoring the vertical settlement, rotation, and translation of the entire substructure as the loads are being applied, vertical displacement at the top of each pile head can be estimated. A suggested failure criterion for the test can be when any single pile head exceeds the deformation limit.

These measurements can be obtained by LVDTs or dial gauges if a stationary reaction beam can be placed near the existing foundation being monitored. Non-contact displacement measurement methods are also suitable if they have sufficient resolution. Recording the rebound and permanent settlement that occur during each test increment helps determine exactly where failure of a pile or the system occurs. Test loads are typically applied in increments with each increment being determined in accordance with standard pile testing guidelines.

Interpretation of Results

Unlike a typical pile load test, a future load test determines the total capacity of a foundation system for reuse, without testing to failure. The capacities of individual piles will vary from one another and could be below the load tested capacity (on a per pile basis). Current design standards typically consider the likelihood of failure for a single pile, rather than for an entire group. Paikowsky et al. (2004) suggested a reliability index of 2.33 for redundant piles and 3.0 for non-redundant piles, correlating to a probability of failure of approximately 1 percent and 0.1 percent, respectively. Davis et al. (2018) present two methodologies that can be used to determine individual pile capacity from a future test load applied to multiple piles. Both methodologies consider the future load to be the minimum possible mean pile capacity. Both methods require a variance between the capacity of individual piles within a population (group of piles driven in similar conditions with similar criteria) be assumed by the designing engineer. Typical values of in-site variance can be taken from Paikowsky et al. (2004) as 0.15, 0.25, and 0.35 for low, medium, or high variance sites. In general, assuming a higher variance will produce more conservative results.

The first methodology updates the resistance factor, ϕ, that is used to find nominal capacity from static or dynamic design equations. The probabilistic distributions of individual pile resistance provided by Paikowsky et al. (2004) are updated by considering that an individual pile is unlikely to have a capacity much lower than the average pile capacity. The distribution of pile capacity is updated using this knowledge, as shown in figure 59 for two levels of future test loading (20 percent of nominal pile capacity and 30 percent of nominal pile capacity) for a specific pile distribution.

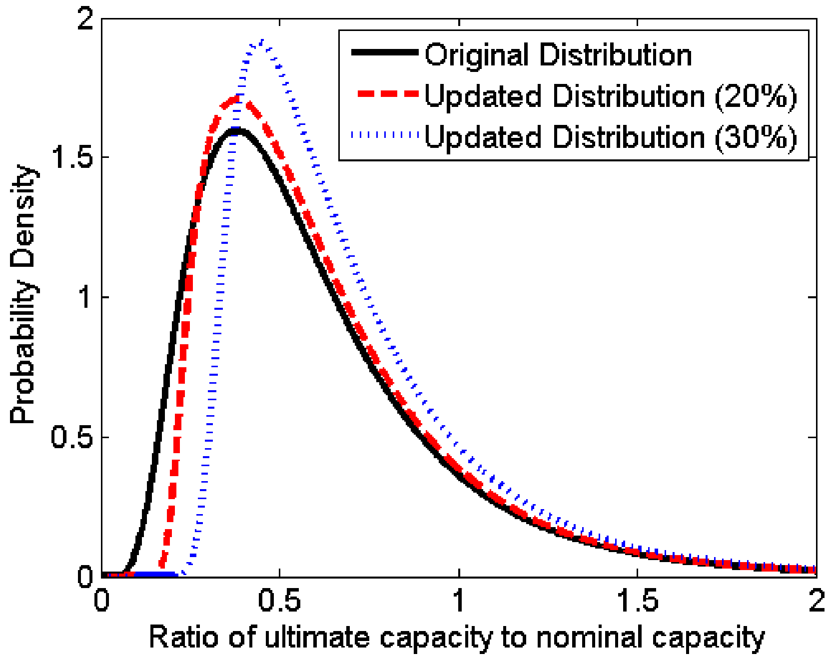


Figure 59. Graph. Original and updated distribution of pile capacity

Reliability analyses are then performed with these updated distributions. The updated distributions have a lower probability of low resistance piles, so as the future test load increases, the resistance factor can also be increased while maintaining a constant reliability. Figure 60 shows the impact this updating has on the resistance factor for piles with their capacity determined by the α-method, as a function of the future test load.

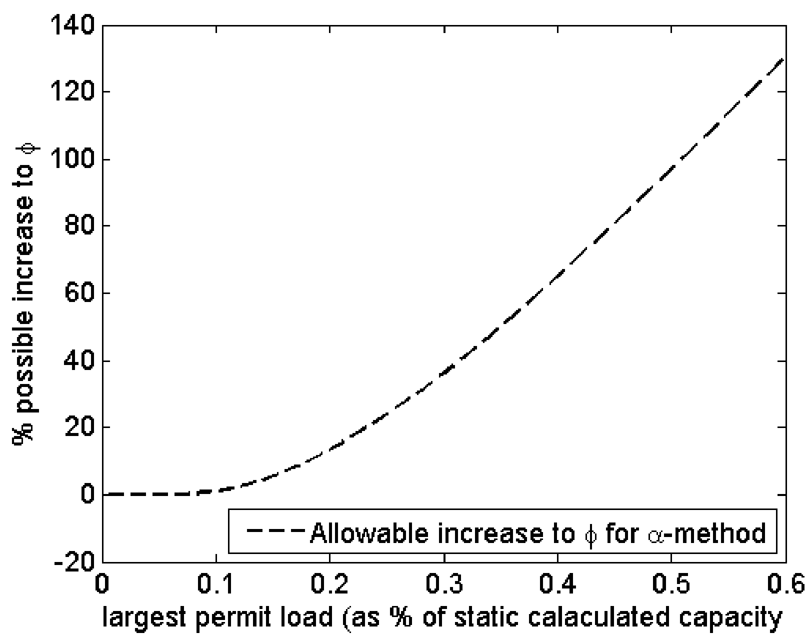


Figure 60. Graph. Resistance factor vs future load for α-method

When piles are end bearing on rock, or do not have design equation predicted capacity, there is no distribution to be updated, and no pertinent resistance factor. For these cases, a methodology is proposed that determines the live load capacity of the piles as a function of the total future test load applied to the piles. This methodology accounts for dead loads, including those removed and replaced during reuse. The capacities of the individual piles are then a function of the amount of dead load removed, amount replaced, and the maximum applied future test load. Figure 61 shows a plot of the available live load capacity for individual piles when future load is applied as a known live load. Figure 61 assumes a low variance site where the dead load is the same before and after reuse, although some is removed and replaced. Due to the uncertainty of any removed and replaced loading, the amount of live load capacity available is dependent on the magnitude of the test load in comparison with the existing dead load. Similar charts for sites of other variances are available in Davis et al. (2018[b]).

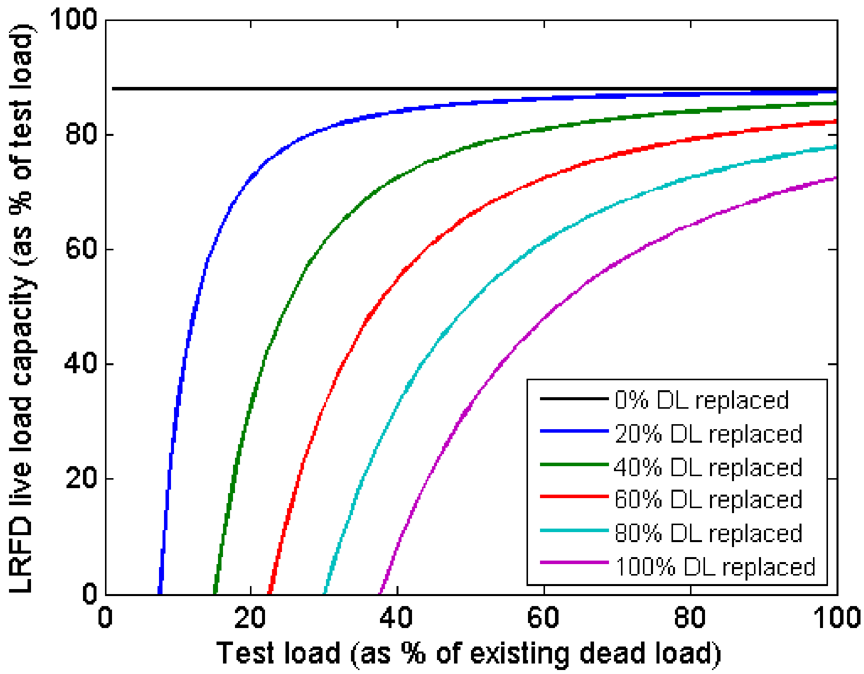


Figure 61. Graph. Capacity vs. future test load for various changes to DL

Foundation Testing with Strain Gauges

For pile bents and column piers on drilled shafts, large sections of pile are exposed above ground and are generally accessible for monitoring. This allows for the installation strain gauges directly onto the elements to estimate the actual load being transferred to the pile or drilled shaft. Timber piles will have a highly variable modulus of elasticity, and the future test load being transferred to timber piles cannot be reliably estimated from strain measurements alone. The strains measured during this stage will be from the future test load only, the dead load contributions from the pile bent and superstructure cannot be directly measured through this type of instrumentation.

There are two general approaches which can be used to estimate the load transferred to each pile from the future test load. The first approach uses basic statics and mechanics principles to determine the axial force and moment at a single elevation. Assuming an element is behaving linear-elastically, the axial strain, ε_z, measured at any exterior location is given by equations (29) and (30):

$$\sigma_Z = \frac{P}{A} \pm \frac{M_Y x}{I_Y} \pm \frac{M_X y}{I_X} \tag{29}$$

$$\varepsilon_z = \frac{\sigma_z}{E} \tag{30}$$

where P = axial force, A = Cross sectional area, $M_{x,y}$ = Moment of inertia (gross) around x and y axes, x, y = distance from x, y centroid to strain gauge and E = Modulus of elasticity.

The total axial force, *P,* at any one location can then be determined by placing two strain gauges on opposite sides of the column, averaging the results, and multiplying by *EA*. The moment acting in that direction can be found by taking the difference in strain gauge measurements and multiplying by $\frac{I}{2x}$, where *I* is the moment of inertia in that direction and 2*x* is the distance in between the strain gauges (diameter or width of column). Placing four strain gauges at the same elevation on orthogonal planes allows for computation of the bidirectional moments and the axial force. Placing this setup at multiple elevations allows for the determination of the shear force induced by the test load, as this shear force will induce a moment that varies linearly with elevation. Dividing the difference in moment by the distance between the strain gauge pairs provides the shear force experienced by the pile.

The second approach is more robust, as it does not assume a modulus of elasticity, cross sectional area, or moment of inertia for the piles being monitored by strain gauges. Using the same general setup above, pairs of strain gauges can be used to measure the axial strain on each pile/column. The total load placed on the superstructure is already known, and thus the relative contribution to each pile can be determined by dividing the average strain on each pile by the sum of strains in all the piles.

The main objective of installing the strain gauges is to establish the magnitude of future test load being carried by each pile. When the amount of load being transferred to each pile can be directly established, the procedure outlined in the previous section can be foregone, and the capacity of each pile can be directly established.

Category 4: Testing on Individual Piles

Static load testing can be used to directly measure load-carrying capacity of a pile using state of the art testing protocols. To perform a static load test, access to a single pile will be required, meaning that a static load test can only be performed when the pile cap can be excavated. The pile selected for testing can be cut to remove a section so that the hydraulic jack can be installed between the pile and the pile cap. The pile cap and pull out resistance of other piles would serve as reaction mass in this case. This procedure can be combined with portions of the integrity assessment, where the condition and intactness of the piles are verified after they are exposed. This method has advantages when excavation of the pile cap would be required for the piles to be examined (checked for deterioration), load tested, or posted. Similarly, steel piles may have experienced corrosion near the pile cap where the pile is exposed to groundwater fluctuations.

This section can be removed, and the remaining pile load tested. Prestressed concrete piles are less suited for testing using this procedure, since cutting them to remove a section may be not be possible because of prestressing tendons.

Prior to excavation, analysis of the foundation can determine whether soil can be safely removed from around the sides of the pile cap. Excavation of the pile cap will cause a small loss in side friction in the exposed pile section, but more importantly can reduce the effective overburden pressure throughout the length of the pile. This procedure will likely require closure of the bridge for the duration of the excavation, load testing procedure, and backfilling.

After excavation, a test pile is identified which is believed to be representative of installed piles. The pile selected for testing will then be cut off several feet below the pile cap. If a portion of the pile has deteriorated, the deteriorated section may be removed at this time, where possible. Once a section is removed, there is room to insert a hydraulic jack that can provide a downward force on the pile, using the mass of the pile cap and pullout resistance of the other piles as a reaction block. The load on the pile can then be estimated by the hydraulic pressure in the jack and with load cell(s) placed above/below the jack. The load applied to the top of the testing pile is then found from the two load cells and the pressure in the jack. The deflection of the head of the cutoff pile is measured using a stationary reference frame. Verifying that the cap and piles have the sufficient capacity and stability to provide the required reaction force against the tested pile will help prevent structural failure during testing. The failure criteria of the tested pile follow standard practice, with the pile being tested until a plunging failure condition is reached. After testing, the connection between the test pile and the pile cap can be restored though posting the pile or inserting a steel section. The excavation around the pile cap can be backfilled with compacted soil, crushed stone or lean concrete. Compaction of soil around the pile may be difficult due to the overhead pile cap.

Another option for testing driven piles involves the testing of newly driven piles adjacent to the existing foundation. For these tests to provide meaningful information on the existing driven piles, the new piles would be of the same geometry and type as the original piles, as well as driven with similar blow count and penetration criteria. This would require pile driving logs for the original foundation installation. This method of determining the existing pile capacity is most useful when additional piles will need to be driven for widening or strengthening. By using identical piles driven to the same criteria, testing on the new piles can be used to determine the strength of the existing piles.

A resistance factor of 0.75 is recommended when only a single static load test is performed for each site condition. This new test result will automatically include the effects of setup and downdrag, which have already occurred. The loss of side friction due to future downdrag and potential scour will reduce capacity below the tested capacity. It is important to note here that the LRFD Bridge Design Specifications (AASHTO 2014) also require that piles be driven with the same criteria used on the test pile. If EOD criteria are not found for the existing bridge piles, a lower resistance factor may be appropriate.

FOOTING AND CAISSON BEARING CAPACITY

In the case studies obtained, the capacity of shallow foundations is most often evaluated in the same manner as current LRFD standards. In many cases, reused foundations were originally designed following working-stress type designs, but completely redesigned for use in LRFD codes. New borings were often used to supplement or confirm old borings, and the capacity was determined from an essentially clean-sheet design of a foundation with the existing dimensions. While borings are typically performed near the existing foundations for soil classification, they can also be performed through the foundation into the underlying soil. This allows for direct sampling of the foundation concrete, the interface between the foundation and the soil/rock, and the soil/rock directly underlying the foundation.

Load testing of existing shallow foundations is also possible through the application of heavy weights, heavy trucks, or loading directly on the foundation. In theory, if sufficient loading was placed on a foundation, the actual bearing capacity of the foundation could be determined. In practice, it may be difficult to subject the entire foundation to sufficient loading that proves the foundation has the required vertical and eccentric vertical capacity. The authors are currently unaware of studies conducted to date that establish the appropriate load factors to be applied for shallow foundations that have been load tested for future reuse loads.

Rebound monitoring can also be performed on footings as the load is removed. The measured rebound can be compared with the existing superstructure weight to estimate the future settlement that will occur when the superstructure is replaced.

Modern practice typically avoids the use of shallow footings in areas that are susceptible to scour, although shallow footings may be suitable if they are founded on non-scourable material or the scour depth is shallower than the bottom of the footing. During analysis of the footing capacity, soil that is expected to be scoured by a 100-year flood is not typically considered to provide any lateral restraint to the foundation, which may impact the overall capacity or stability of the foundation if subjected to substantial lateral loads. Contraction scour, as discussed in greater detail in Chapter 8, may also lower the bearing capacity of the foundation by reducing the overburden pressure.

CASE STUDIES

Lake Mary Bridge, near Flagstaff, AZ

Testing was performed on samples extracted from coreholes drilled through the piers and abutments. The quantities obtained from the sampling were sufficient to develop simplified material models for the concrete.

Electrical Resistivity Imaging (ERI), Seismic Refraction Tomography (SRT), and Multichannel Analysis of Surface Waves (MASW) were employed to ascertain the properties of the bedrock and soil underlying the piers. The bedrock was found to have an electrical resistivity of over 50 Ohm-m, while the soil varied between 5 Ω.m and 50 Ω.m.

A finite element model containing approximately 50,000 solid elements was used to model the superstructure and substructure. The model was analyzed with fixed conditions representing the bottom of the pier.

Milton Madison Bridge, between Milton, IN and Madison, KY

Design Properties

The unconfined compressive strength of the concrete was taken to be 6,580 psi (45.4 MPa), 1.5 standard deviations below the average compressive strength from the 54 tests. The static modulus of elasticity was estimated from equation 5.2.4.2-1 of the LRFD Bridge Design Specifications (AASHTO 2014) and was found to be 686,730 ksf (32.9 GPa). The original pier had been lightly reinforced, and corrosion or the reinforcement steel was noted in some places, as discussed in Chapter 4. From the design plans, the caissons were unreinforced, and the dowels connecting the pier to the caisson were inadequate. As such, the pier was initially analyzed as an unreinforced section of concrete, and the existing rebar was largely ignored.

Soil and Rock Analysis

A Hoek-Brown model was used to determine the nominal bearing resistance of the rock surface using the Rock Mass Rating (RMR), and results from unconfined compression testing. A nominal bearing capacity of 75 ksf (3.6 MPa) was established, and parameters for use in a Mohr-Coulomb model were determined. The loading imparted on the soil was determined through finite element modeling with the parameters obtained during testing.

Scour Analysis

Predicted contraction scour was up to 3.0 ft (1 m), while predicted local scour was up to 41.1 ft (12.5 m) deep. Finite element modeling showed that the pier was not adequate when the soil alongside the piers was subjected to scour. The existing caisson was deemed inadequate for large amounts of scour, as the eccentricity and bearing pressure limits were both exceeded. As discussed in Chapter 7, preventative measures were taken to prevent scour from occurring.

Modeling of Existing Conditions

The corrosion observed on sections of the existing reinforcement combined with the very low reinforcement ratio, led designers to evaluate the piers are unreinforced concrete sections. The lift lines and locally weaker concrete gave some concerns about the continuity of the sections, although the lift lines were horizontal and the compression in the piers at the liftlines would likely prevent shear failure of that portion. The caissons were also unreinforced, aside from various bars doweled into the top of the caissons to connect them to the piers.

Finite Element Modeling (FEM) was performed with Midas GTS of the pier stems, caissons, and soil systems. Originally, the piers were evaluated without considering the passive resistance of the side soil, as would be typical for spread footings bearing on rock. The FEM solutions indicated that the eccentricity was greater than B/2 for many of the foundations investigated, meaning that they could not be in static equilibrium (entire foundation in uplift). For the load cases and piers where the eccentricity did not exceed B/2, the bearing stresses all exceed the 75 ksf (3.6 MPa) capacity calculated for Extreme Event limit cases, or the 33.75 ksf (1.6 MPa) calculated for Strength limit cases. Even without the resistance of the side soil being considered, the footings were deemed adequate against sliding assuming a coefficient of friction between the soil and rock of 0.75. The foundations were then reanalyzed assuming no loss of soil, something that would require scour countermeasures to be applicable. The reanalysis assumed that active pressure developed on the side that the pier moved away from, and that passive resistance was developed on the side the pier moved toward. A factor of 1.5 was placed on the active pressure (AASHTO 3.4.1), and a factor of 1.0 was placed on the passive pressure (not supplied in AASHTO for overturning of footing on rock). This loading was included on another model run that showed the eccentricity of the footing remained below B/6 and the maximum bearing stresses were below the 75 ksf (3.6 MPa) and 33.75 ksf (1.6 MPa) limits for Extreme and Strength limit states, respectively.

Three-Dimensional FEM was employed using a Mohr-Coulomb failure envelope for the surrounding soil and underlying rock. The concrete was modeled using elastic solid elements. An allowable limit of 95 psi (6.6 MPa) (15 percent of the modulus of rupture) was placed on all Strength and Extreme Event limit states, while no tension was allowed for all Service limit states. Both of those requirements are typical for unreinforced concrete. The section was analyzed assuming various types of bearings, including elastomeric bearings with varying stiffness. The Strength III, Service I, Service IV, and Extreme II load cases were all exceeded for the pier stems and caisson located in the river, existing Piers 6, 7, and 8. Existing Pier 9 underwent tension in the Service I and Service IV load cases. The various bearing pad types investigated all produced similar results that showed the existing columns were insufficient for the proposed new loads. If soil strengthening were to be performed, the tensile loads in the piers could be reduced, however it was determined that soil strengthening could not be performed in conjunction with scour countermeasures.

North Torrey Pines Bridge, Del Mar, CA

Design Properties

The unconfined compressive strength of the concrete was taken to be 4200 psi (29 MPa), based on the compressive strength test results. The failure strain of concrete used during the non-linear analysis was assumed to be 0.005. The yield strength of rebar was taken to be 46 ksi (317 MPa).

Ground Motions and Displacement Demands

Ground motion resulting from seismic events were a major source of loading considered on the foundation. Originally, the bridge had been analyzed based on curves from (Caltrans 1996). The site had peak ground accelerations of 0.6g, and a site class of D. Site-specific ground motions were determined with Next Generation Attenuation (NGA) relationships that drastically lowered the expected ground motions at the site over the curves. The displacement demands were formulated using the three site-specific ground motions investigated.

Modeling of Existing Conditions

The capacity assessment of the original structure was performed with a SAP2000 model of the as-built drawing made during evaluation. The bridge was initially analyzed assuming that corrosion had not impacted the capacity of the piles bents. Non-linear elasto-plastic hinges were input at areas of stress concentration and expected hinge formation. The failure of the hinge was defined as the amount of curvature required to cause a compressive strain of 0.005 in the concrete. Non-linear P-y springs, determined from the geotechnical analysis, were used to model the soil behavior. Changes to the soil springs due to liquefaction were not considered, as it was assumed these conditions would be mitigated prior to reuse.

A pushover analysis was performed to determine the displacement capacities of the elements, as well as the shear and moment demands on the elastic components these displacements. The analysis showed structural deficiencies with displacement capacity, shear capacity of the cap beams and columns, and yielding of transverse bents. The progression of failure for each pile bent was found and summarized.

Liquefaction and Slope Stability

The soil analysis showed several areas vulnerable to the potential for liquefaction. Some areas were near bridge bents and could cause substantial changes to the soil behavior under that bent. Liquefaction also impacted the slope stability analysis, as portions of the slope had liquefiable areas.

The slope stability analysis found that the slopes were stable under the static analysis, as expected due to the age of the existing conditions. Liquefaction was included in slope analyses where the soil was identified as potentially liquefiable. Without mitigation, liquefaction induced slope stability issues could cause a slope failure with up to 10 ft (3 m) of soil displacement. The slope failure would likely impact the bridge and cause severe damage if left unmitigated. A slope stability analysis was also performed assuming the liquefaction had been mitigated, but still included seismic loadings. Up to 40 inches (1 m) of soil displacement was expected under this loading, and it was determined that ground strengthening to mitigate this risk was also needed.

Georgia Street Bridge, San Diego, CA

The concrete compressive strength for the arch ribs was taken to be 3000 psi (20.7 MPa), lower than any of the testing performed. The thrust blocks were analyzed as being cast directly against the weak sandstone formation. A bearing capacity of 12 tsf (1.15 MPa) was assumed for the thrust blocks and found considering the rock classification performed from samples obtained from boreholes. The capacity of the anchor blocks was determined from the original design plans. The new wall face was doweled into the existing anchor block, so the capacity of the elements connecting the wall and the anchor block was that of a newly designed section.

Due to the poor condition of the abutment walls noted during the integrity and durability analysis, it was determined the walls could not be relied upon for capacity.

Huey P. Long Bridge, Jefferson Parish, LA

Design Strength of Materials

The original construction records provided test break data for concrete found in both the piers and the underlying caissons. While the design strength was 2,000 psi (13.8 MPa), the test breaks were found to be significantly higher, at 3,800 psi (26.2 MPa) for the piers, and 3,300 psi (22.8 MPa) for the caissons. The findings of ACI Committee 209 (ACI 1992) and of A.M. Neville (1995) were employed to estimate the current compressive strength based on the 28-day compressive strength reported in the 1930 test breaks. The current compressive strength was estimated to be 4,500 psi (31 MPa) and 8,800 (60.7 MPa) psi for the pier using ACI (1992) and Neville (1995), respectively. The compressive strength for the caisson concrete was found to be 3,870 psi (26.7 MPa) and 7,565 psi (52.4 MPa) following ACI (1992) and Neville (1995), respectively. The design compressive strength used for analysis of the pier was 4,000 psi (27.6 MPa), and the design compressive strength used for the caisson was 3,800 psi (26.2 MPa). Still, even considering the increased strength, widening of the piers was required to support the wider steel truss sections of the reused piers.

Bearing Capacity

The bearing capacity for the sand layer underlying the caisson was originally investigated by Dr. Karl Terzaghi in 1926. These original calculations determined that an allowable bearing capacity of 5.5 tsf (527 kPa) would be acceptable for these foundations. The ultimate loadings applied to the bearing strata were 2.8 tsf (270 kPa), 2.6 tsf (250 kPa), 1.9 tsf (180 kPa), and 1.4 tsf (130 kPa) for Piers A, I, II, III, and IV, respectively.

An exploration was performed to support the conclusion that 5.5 tsf (527 kPa) was a safe bearing capacity for the existing caissons. After performance and review of borings, a net ultimate bearing capacity of 43.1 tsf (4.1 MPa) was determined for the caissons. This far exceeds the original criteria of 5.5 tsf (527 kPa), which was deemed to be adequate for the proposed additional loads.

Settlement

Approximately 3.5 inches (90 mm) to 5 inches (230 mm) of settlement was predicated at the bridge piers prior to original construction. No detectable settlement had been noted from measurements taken between 1940 and 1990. The observed settlement correlated well with the settlement expected from the initial exploration. Based on the findings of the investigation, estimated settlements from the additional load were expected to be in the range of 1 to 1-½ inches (25 to 37.5 mm) from the effect of the newly imposed additional loading. Surveys of the piers during construction did not detect any additional settlement.

NJ Route 72 Bridges, Ocean County, NJ

Pile Capacity

The capacity of the existing timber piles was determined from the measured dimensions and the allowable stresses prescribed for the identified wood species. The piles were initially analyzed considered that existing scour is repaired to check that piles had sufficient capacity in the non-scoured condition. Piles with section loss were analyzed considering this section loss for axial capacity calculations. For stability calculations, any piles with sections that were expected to be repaired were analyzed considering their full dimensions, by assuming that the pile jacket repairs would restore any lost stability. The piles were assumed to terminate in the bearing stratum, as NDT had confirmed the minimum depth and the piles had a history of adequate performance.

During normal operation (without scour), several isolated piles were found to be overloaded, experiencing stresses beyond their ultimate capacity. Overstressed piles typically had small original diameters and/or high section loss, leading to insufficient cross-sections. An analysis was performed to determine if the piles adjacent to the overstressed piles had sufficient capacity to take up the overstress loads, assuming they could be transferred away from the overstressed pile. The analysis showed that pile on either side of each isolated overstressed pile had enough reserve capacity to take their original loading, as well as the entire overstress loading on the overstressed pile.

Scour

The anticipated scour depths were determined in accordance with HEC-18 (Arneson 2012) for both the 100-year flood and the 500-year flood. The 100-year flood scour event was analyzed with full dead load and 50 percent of the live load, while the 500-year flood was analyzed with full dead loads and no live load. The analysis found that all bents were stable under the 100-year flood, although some piles became structurally overloaded. The piles adjacent to the overstressed piles had sufficient reserve capacity to carry the overstress loads, much like during the non-scoured pile analysis. A similar scenario occurred during the analysis of the 500-year flood, although several bents were found to also be unstable during the 500-year flood. Scour countermeasures are required to make these bents stable during the 500-year flood.

U.S. Route 2A Bridge, Haynesville, ME

Driven Pile Test Program

Initially, the capacity of the in-situ piles was estimated static calculations (Nordlund method for side resistance, Thurman method for tip resistance). When combined with the resistance factors from AASHTO (2014), there was insufficient capacity for the LRFD analysis. A test program was devised to take advantage of the higher resistance factors prescribed by AASHTO (2014) for piles where representative testing has been performed. A static load test was conducted under each abutment, with the capacity for each abutment being the respective failure load with a resistance factor of 0.70. The lower failure load along with a resistance factor of 0.65 was used for the center pier.

The load testing was performed by excavating a portion of the pile cap and exposing the timber piles. After visual inspection, a section was removed with a chainsaw and a Pile Integrity Test (PIT) was performed. A hydraulic jack was placed in the section removed from the pile. The mass of the concrete pile cap and the superstructure was used as a reaction force for the hydraulic jack. The piles were tested until a plunging failure was observed, and the pile failure criterion was defined with Davison's criterion.

U.S. Route 1 Viaduct, Bath Maine

Pile Capacity

The primary concerns regarding the capacity of the steel H-piles was ensuring the piles had been driven to bedrock and that significant corrosion had not occurred. During the integrity assessment, pile driving logs were compared to new subsurface explorations to confirm that the recorded depths roughly matched the expected bedrock surface level. Parallel seismic was performed to confirm that piles had been driven to the depths on the driving record. It was noted that there was a potential for very hard driving during installation due to the stiffness of the overlying soil layers.

Since virtually no corrosion was noted on select piles during test pit excavation there was not much concern that the pile had already suffered deterioration. The capacity of the existing piles was calculated using a reduced cross-section to account for the possibility of existing corrosion and potential future corrosion. 1/16 inches (1.6 mm) was recommended to be removed from the cross section of the pile for analysis.

The existing plans and design information had led the engineering team to conclude that the piles had been driven to practical refusal on bedrock. The available capacity for these piles was therefore governed by the structural capacity of the pile itself. The original capacity of 92.6 kips/pile (412 kN/pile) was based on an allowable strength design with a factor of safety of 5.5 being placed on the yield stress of the steel. In effect, this meant that the pile capacity was $1.25 \times f_y / 5.5$, where the 1.25 was a factor that allowed additional capacity due to the nature of the loading. The original design plans specified ASTM A-7 steel, which would have a 33 ksi (228 MPa) yield stress. Buckling was not a concern due to the stiffness of the surrounding soil. The new capacity was determined in accordance with section 10.7.3.2.3 of AASHTO LRFD (2014), based on the structural capacity of the pile. A resistance factor of 0.5 was chosen due to the potential of damage during hard driving. The factored resistance for use in LRFD was found to be 140 kips/pile (623 kN/pile). This does not necessarily correlate to an increase in capacity due to the difference between ASD and LRFD codes.

Jackson Road Bridge, Lancaster, MA

The original pile foundation plans, boring logs, pile driving logs, and hammer information were available from DOT archives. No load testing data was available, although it had been recorded that three load tests had been performed. Since the driving logs were available, the capacity was estimated using the FHWA Modified Gates equation, which relies on end-of-driving criteria to establish capacity.

The loading on the individual piles was determined and the compared with the factored resistance following the LRFD Bridge Design Specifications (AASHTO 2014). The results of the analysis showed that pier piles had sufficient capacity while some of the abutment piles were overloaded. Loading on these overloaded abutment piles was reduced by replacing abutment backfill with lightweight geofoam blocks.

Cedar Street Bridge, Wellesley, MA

To evaluate the geotechnical capacity of the foundations, the results of two sets of subsurface investigations in 2005 and 2010 were reviewed. The 2005 investigation consisted of three borings; one at each abutment and one at the center pier. The 2010 investigation was primarily used to evaluate the bearing capacity at the location of the temporary supports during construction. The bearing capacity was calculated using Terzhagi's equation and performance factor of 0.45, taken from the LRFD Bridge Specification (AASHTO 2014). The sliding, overturning, and bearing capacity of the abutments were determined using the soil properties of the backfill and Coulomb Earth Pressure coefficients, while ignoring passive earth pressure at the toe. The calculations showed the replacement bridge would have adequate capacity for the proposed loads. Since the net change in loading on the foundation was small and the soil was deemed competent, there were no anticipated settlement problems. Therefore, the existing foundation was reused without any modification.

Crowchild Trail Bridge, Alberta, Canada

FLAC 3D was used to evaluate the loading and capacity on the new foundations. The parameters used in the modeling for rock and soil characteristics were derived from the completed boreholes. The primary geotechnical consideration for the reused bridge is the potential of settlements due to the increased loading on the riverine piers. The FLAC analysis concluded that the additional settlement due to the increased loading will be in the acceptable range. The land-based piers will be widened as necessary to reduce the likelihood of settlement for these piers that bear on gravel footings. Monitoring of the bridge will be performed during construction to confirm that any additional settlement is within the acceptable rang

CHAPTER 7. INNOVATIVE MATERIALS AND FOUNDATION ENHANCEMENT

INTRODUCTION

The integrity, durability, and capacity assessments provide a list of deficiencies associated with the substructure being proposed for reuse. Most often there will be overlapping concerns identified in the three segments of evaluation that need to be addressed prior to reuse. The selection of foundation strengthening measures considers the issues identified during the integrity, durability, and capacity assessments to produce an acceptable reuse design. In some cases, strengthening/repair options considered for one issue will also impact other identified issues.

Based on NCHRP Synthesis 505 (Boeckmann and Loehr 2017) and review of case studies in Chapter 1, the following options for foundation strengthening are available:

- Increased footing size (applicable for shallow foundations for improvements in structural and geotechnical capacity of footings as well as vulnerability to hazards, such as seismic)
- Additional deep foundation elements: Load capacity of existing foundations can be increased by installing micropiles, drilled shafts, driven piles, ground anchors, stone columns or other deep foundation elements.
- Ground improvement:
  - Global ground modification techniques: techniques for improving the strength and stiffness of the overall soil or rock mass into which the existing foundation is installed. This will improve the bearing capacity of foundations (both deep and shallow) and can address issues such as liquefaction during seismic hazards.
  - Local ground improvement techniques: techniques that are more narrowly targeted toward increasing the side and/or tip resistance of the individual deep foundation elements such as a shaft. For example, the RuFUS manual (Butcher et al 2006) recommends grouting to improve side resistance and jet grouting near the base of a deep foundation to improve tip resistance.
- Strengthening of above ground foundation elements: Increasing the load capacity of above ground elements through pier stem widening, addition of tiebacks, replacing backfill with lightweight fill to reduce loading on abutment, soil nails, encasing of pile bents by concrete or FRP, wall encasement of piers/pile bents.

Figure 62 shows the results of the NCHRP Synthesis 505 (Boeckmann and Loehr 2017) survey on the use of foundation strengthening methods by State DOTs and other agencies. It can be observed from figure 62 that seven different construction techniques have been used by at least one-quarter of respondents. Most commonly used strengthening methods are the addition of driven piles (73 percent), increasing footing size (63 percent), and addition of micropiles (52 percent).

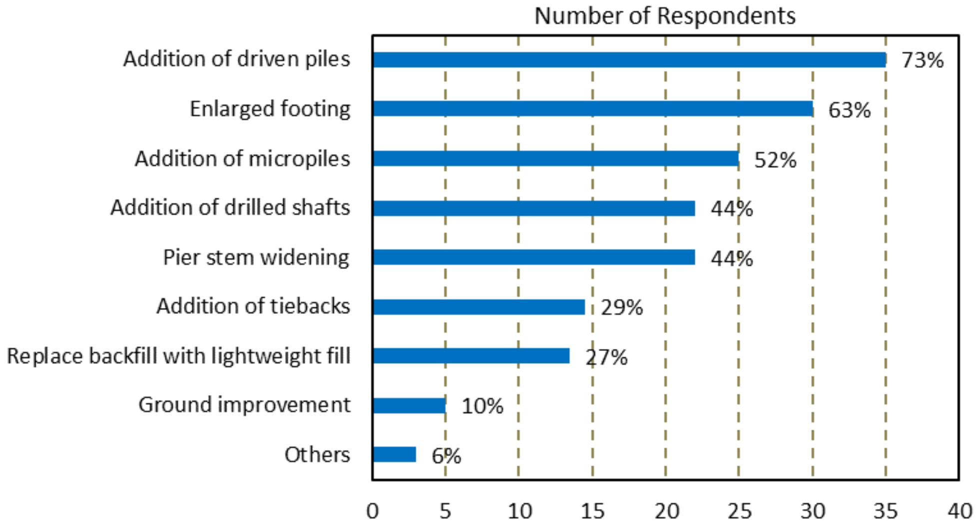


Figure 62. Chart. Survey on foundation strengthening methods

Wan et al. (2013), based on the outcome of a nationwide survey on repair techniques of substructure elements, have presented substructure repair options for concrete, steel and timber elements and their expected service lives, as shown in table 32.

Table 32. Repair techniques for substructure elements (Wan et al. 2013)

	Repair Methods	Timber	Steel	Concrete	Service Life (Years)
Pile Repair	Pile Posting	×			
	Concrete Encasement	×		×	20
	Pile Restoration	×			
	Pile Augmentation	×			
	PVC Wrap	×			
	FRP Wrap	×		×	50-75
	Pile Shimming	×			
	Adding Steel		×		
	Pile Jacket		×	×	20
	Anodes		×	×	15
	Anode Embedded Jacket		×	×	10-35
Supplemental Piles	Steel Piles		×		
	Timber Piles	×			
	Concrete Piles			×	

STRENGTHENING AND REPAIR OF STRUCTURAL ELEMENTS

Concrete Elements

ACI 546.1R (ACI 1997) discusses available reinforcement and repair techniques for concrete members. Table 33 shows commonly used strengthening measures for concrete foundation elements. A detailed description of some of the prominent strengthening measures for concrete elements is also presented in the following.

Table 33. Strengthening and repair option for concrete elements

	Identified Issue	Strengthening Measures Available
Integrity Issues	Concrete Damage	Demolition and replacement of impacted concrete
	Alkali Silica Reactivity	Removal and replacement of impacted concrete, replacement of ASR impacted members
	Corroded Reinforcement (loss of rebar area)	Doweling, external rebar placement, FRP wrapping
Durability Issues	Chloride Ingress	ECE, removal and replacement of affected concrete, cathodic protection
	Spalling/delaminations	Repair of spalled areas, placement of anodes in repair to prevent corrosion, wrapping of affected areas, addressing primary issue causing spalling
	Freeze-Thaw	Removal of impacted concrete and replacement with HPC, wrapping of vulnerable concrete with moisture barriers
	Carbonation	Remove/replace carbonated concrete, wrapping with barriers to prevent moisture and CO_2 exposure, cathodic protection to prevent corrosion
Capacity Issues	Increased Loads	Addition of new elements, encasement of existing concrete sections, addition of external reinforcement cage, FRP wrapping, doweling of additional bars,
	Low Concrete Strength	Replace/add elements, encase with new concrete
	Under-reinforcement, detailing issues	Doweling, encasement with additional reinforcement cage, FRP wrapping of low capacity sections

Patching Spalled and Cracked Concrete

Weyers et al. (1993) have discussed patching techniques for substructures and superstructures. The two main patching techniques discussed are patching with Portland cement concrete (PCC), and shotcrete. PCC patches have an expected service life of 5 to 10 years, while shotcrete has an expected service life of 10 to 15 years, assuming the underlying problems (sources of corrosion) are left uncorrected. Service lives in these ranges are generally unacceptable for reuse, as the expected service life for new bridges can be 50 to 75 years. The longevity of an individual patch may be greatly increased by installing cathodic protection, but if the underlying problem of chloride contamination or carbonation has not been addressed, surrounding areas can begin to corrode. When spalling is caused by a small patch of corroding steel, this patch can act as an anode and essentially provide cathodic protection to the surrounding steel. The installation of galvanic anodes can interrupt this macro cell and allow corrosion to begin in adjacent areas where spalling had not occurred previously.

If patching and replacement is considered as an alternative for substructures impacted by spalling, there may be costs associated with future repairs of the substructure. Performing an extensive analysis into the remaining service life of these repairs allows for an accurate portrayal of the associated life cycle costs.

Cover Replacement and Encasement

When concrete is substantially impacted by chlorides or carbonation, removal of the contaminated cover concrete may be necessary to prevent corrosion of the reinforcement. The cover concrete can be removed through methods such as chipping, hydrodemolition, or hammering. Removal typically includes all concrete impacted by the contamination, even potentially deeper than the reinforcement layer. High quality replacement concrete will be an appropriate mixture for the bridge environment, with the proper type and amount of air entrainment. Connectivity to the underlying concrete is often provided to prevent a potential delamination along the joint between old concrete and new concrete. Typically, this involves doweling hooked bar or hoops into the original section and casting the new concrete around the bars. Not providing a smooth finish to the existing concrete after removal allows for additional mechanical interlocking between the two concrete layers. The elements can be increased in size at this time to provide additional capacity. One important consideration is whether the original steel corrosion is halted prior to placement of new concrete. If rebar is still exposed to chloride contamination or carbonated concrete, corrosion will continue in these bars and the resulting expansive forces will damage the new concrete layer.

Two major options are available for replacement of the cover concrete: (i) placing new forms on the outside of the existing column and pouring concrete, (ii) shotcrete. Structurally, the two methods will produce comparable results. The construction sequence for formed concrete will require more labor in the form placement, but the actual concrete placement will be a simpler process. Shotcrete does not require forming and can be used on very tall structures where concrete would otherwise be poured in multiple lifts.

FRP Wrapping

Fiber-Reinforced Polymer (FRP) wrapping is a strengthening method being increasingly employed for concrete element strengthening, protection, and seismic rehabilitation (Wan et al. 2013). The FRP wrap has several important benefits for the existing concrete: it prevents further intrusion of chlorides and chemicals, it provides tensile reinforcement on the exterior of the element (longer moment arm than with reinforcement and cover), it provides confining stresses to concrete columns (improving concrete behavior), and it prevents patches and repairs from spalling off the element. FRP wrapping can be given a smooth appearance that hides patches that may not be the same color as the base concrete. NCHRP 655, *Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (Zureick et al. 2010) discusses repair of concrete elements with FRP.

Reinforcing Earth Retaining Structures

Many earth retaining structures have reinforced concrete faces that can become exposed to chlorides or carbonation. Replacement of a section of such structures is complicated by the necessity to continue retaining soil during construction. Depending on the soil being retained, it is sometimes possible to deconstruct and replace the retaining wall in segments, but in practice this is generally costly and time consuming. Another method would be to install sheet piling or soldier piles behind the existing wall, then deconstruct the existing wall and build out the new wall, working from top to bottom. This method requires appropriate soil conditions behind the wall and is helped if the soil is easy to drill through, and if it can retain a vertical cut for several feet to aid in demolition of the existing wall. Another option is to place a new concrete face in front of the existing wall. This allows for the installation of entirely new concrete and rebar. This new face can provide the entirety of the required retaining capacity, with the existing wall being simply abandoned in place. The new wall can be made contiguous with the existing wall or soil system using tiebacks, soil nails, or shear transfer bars.

Doweling

When inadequate reinforcement exists in the existing construction, it may be desirable to add additional reinforcement to the original section. This is achieved by a process referred to as "doweling," where a hole is drilled in the original concrete, reinforcement is inserted, and the hole is filled with grout. There are several important limitations to this technique: starting drilling may be difficult without a flat surface roughly perpendicular to the dowel, there may not be enough space for appropriate development length, and the concrete being doweled may be chloride contaminated or carbonated causing the new reinforcement to undergo corrosion.

Cathodic Protection

Corrosion of the reinforcing steel can be inhibited by forcing the rebar to remain in its cathodic state. In a normal corrosion cell, a portion of rebar will act as the anode, where the formation of rust frees electrons, and a portion of rebar will act as a cathode, where these electrons combine with water and oxygen to produce alkalinity. The alkaline environment passivates the cathode and prevents the formation of rust at that location. Cathodic protection refers to a system that exploits this behavior by forcing the reinforcement steel to be the cathode and therefore preventing corrosion. There are two broad types of cathodic protection systems: impressed current and galvanic. Impressed current systems require continuous connection to a power source that provides a charge differential between the reinforcement being protected and anodes (often metal meshes) placed nearby. Galvanic systems use sacrificial anodes connected to the rebar in the areas where protection is required, like previously repaired spalls or areas of observed corrosion. A more complete overview of how cathodic protection systems work is given by Islam and Daily (2006), Bertolini et al. (2013), Mehta and Monteiro (2014), and Dugarte and Sagüés (2009).

Impressed current systems allow greater control of the cathodic protection, but at increased cost and complexity. The mesh anodes typically employed for impressed current systems on bridge decks are difficult to use on substructure elements, so generally only galvanic protection can be employed for substructures. Both impressed current systems and galvanic system will incur additional life cycle costs over the life of the element.

Electrochemical Chloride Extraction and Realkalisation

Like cathodic protection, Electrochemical Chloride Extraction (ECE) and Electrochemical Realkalisation (ECR) use impressed currents to force chloride out of the concrete and realkalize the concrete around steel, respectively. For these techniques, the anode is often a metallic mesh in an electrolytic solution placed on the concrete surface. The surface anode will draw negatively charged chloride ions toward the surface and eventually out of the concrete. The rebar acting as a cathode produces hydroxide ions that repassivate the concrete surrounding the rebar, protecting the steel and reversing potential effects of carbonation. This technique is discussed by Islam and Daily (2006), and Bertolini et al. (2013).

In general, this technique is best suited only for short life extensions (on the order of 5 to 10 years). While removal of the chlorides and repassivation protects the steel, the underlying issue, either chloride exposure or carbonation, is not addressed by this technique. For ECE to be successfully employed in a long-term design, other measures to prevent the underlying problem are typically used simultaneously. Additional actions, like encasement of the element in fresh concrete, wrapping the element with waterproof membranes, or superstructure changes that prevent surface runoff can prevent the long-term exposure to chloride that created the initial problem.

Steel Elements

Steel components undergoing corrosion will require repair to prevent future corrosion leading to excessive section loss. Browne et al. (2010a) have provided recommendations on repair options for deteriorating steel piles in underwater environments, as shown in table 34.

Table 34. Possible repair methods for underwater steel piles (Brown et al 2010a)

Damage	Repair Option
No Visible Deterioration	Coatings, Pile Wrap
<15% section loss	Pile Jacket
15%-30% section loss	Pile Jacket with Reinforcement
>30% section loss	Partial Replacement

Encasement, Jacketing, and Wrapping

Like concrete piles, steel piles can be encased or jacketed in concrete, or wrapped with a waterproof membrane. In general, these techniques are performed on steel piles for the primary purpose of creating a moisture barrier and preventing future corrosion from occurring. Concrete encasement of steel piles can allow for additional strength if analyzed as a composite section. Encased piles will be stiffer than non-encased piles.

Paint & Coating Repair

Piles being analyzed for reuse may have been coated during original installation. Often, the paints or coatings used to inhibit corrosion may become worn down or may lose effectiveness over the course of a foundation’s lifespan. If the piles are still in good condition, repainting or recoating the pile may be possible. If deterioration has begun or painting is not possible, encasement of the element or wrapping of the element may be needed to prevent further corrosion.

Cathodic Protection

Cathodic protection systems can be installed on underground or underwater steel elements, although they are not generally as effective against atmospheric corrosion. Like concrete elements, both impressed current and galvanic systems can be used. Galvanic systems rely on sacrificial anodes that will require regular replacement. Impressed current systems will not require anode replacement, but there is a greater amount of equipment related to the system that may require repair or maintenance.

Timber Elements

Posting and Splicing of Piles

Portions of timber piles above the ground surface or the groundwater line tend to deteriorate, whereas submerged portions can have an infinite lifespan. When deterioration is only observed in these above ground portions, it is common to replace the deteriorated portions with new timber, steel, or concrete elements. While various techniques exist, the basic procedure involves cutting above and below the deteriorated section, removing the deteriorated portion, installing a new section, and connecting the new section to the original pile. Various connection details, including concrete jacketing, lap splicing, bolting baseplates into the original pile, and bolting fishplates to the exterior of the new and existing portions are available. A more complete list of the connection options and how to employ them is provided by Dahlberg et al. (2015). Timber piles connected to concrete pile caps may require complete excavation to post the underlying piles.

When deterioration of small sections is observed, but removal and replacement is not feasible or prohibitively expensive, it is possible to splice in sister elements that take the load throughout the section spliced. Removal of the original section is not required, but the original element will continue to decay, potentially outside of the sister element location. Splicing in sister elements can also be performed for sections that do not have adequate flexural or axial capacity, even if deterioration has not been noted.

PVC Wrapping

Wrapping of timber piles in polyvinyl chloride (PVC) is a repair technique to counter durability issues, but not integrity or capacity issues. The PVC wrapping does not provide meaningful additional strength and is instead used solely to prevent the flow of water in or out of the pile. This creates a toxic environment for the biological organisms causing decay, thereby preventing future decay.

FRP Wrapping

Unlike PVC wrapping, Fiber Reinforced Polymer (FRP) wrapping for timber piles will be suitable for increasing the capacity of the section being wrapped. FRP has high tensile strength and is bonded to the exterior of the timber element by a grout or epoxy layer poured inside of the wrapping. Like PVC wrapping, the wrapping prevents the flow of water in and out of the element, inhibiting biological decay. This method is most suited for providing moderate strengthening along with prevention of future decay.

Grout/Epoxy Injection

FRP wrapping alone may not be effective in case of significant decay on internal portions of a timber element. Since timber elements typically decay from the inside out, there may be a shell of structurally sound timber surrounding an interior with large voids. Dahlberg et al. (2015) describe a method of repairing timber elements by injecting grout and aggregate in these voids. For the repaired pile to be effective, FRP wrapping of the exterior portion is usually performed after grout injection. Emerson (2004) performed testing on piles repaired using this technique and found that the repaired piles had strength at least equal to the strength expected for an intact timber pile. The combination of grout injection and FRP wrapping restores capacity losses due to findings from the integrity analysis, while preventing future deterioration potentially noted during the durability analysis.

Jacketing/Encasement

Encasement by concrete is one of the most commonly employed repair techniques on timber elements. This method is generally the simplest repair technique, although not necessarily the most inexpensive. Instead of filling the internal voids created by decay, additional concrete, possibly with reinforcement, is placed outside the timber element to increase the capacity of the section. The encasement is done by placing a form (i.e. corrugated steel) of the desired diameter around the pile, inserting rebar and pouring concrete inside the form. Reinforcement can be doweled into the pile to provide a better connection between concrete and timber.

Masonry Structures

Encasement

Like concrete and steel elements, masonry elements can be encased in concrete. The encasement of the pier in concrete can prevent continued spalling or degradation of the exterior layer of masonry blocks or grout. Concerns about shear capacity, ductility, or tensile capacity can be addressed by including longitudinal or shear reinforcement in the exterior layer. Reinforcement can be doweled into the original masonry to provide continuity with the encased concrete.

Compaction/Injection Grouting

When voids due to mortar loss, shifting of blocks, or block degradation are discovered during the integrity assessment, the masonry section may need to be repaired prior to reuse. One approach to repair masonry sections is to drill a hole into the masonry, usually from the top surface, and inject low slump grout under high pressure. The pressurized grout fills interior voids with sound mortar.

Addition or Replacement of Structural Elements

Often, the original structure does not have the required carrying capacity for new loading, and additional elements need to be installed to resist the new loading associated with reuse. Possible changes than can be made include pouring additional concrete adjacent to a pier, adding new piles or columns, or constructing entirely new piers to supplement existing piers and accommodating shorter or different span lengths. It is important when analyzing new foundation elements to consider the loading sequence and the behavior of both the existing and newly installed elements.

As loading is removed from existing elements, it will not rebound as much as the loading initially caused it to settle. Due to this effect, the original elements will be substantially stiffer than the replacement foundation elements and will take up a disproportionate amount of the initial loading. These effects can often be minimized through appropriate construction sequencing or using jacks to preload the additional elements.

Seismic Retrofit

Many older foundations were not designed to the earthquake loading standards present in modern codes. New analysis may show components experience substantially higher loads or displacement demand from seismic activity than they were originally designed for. In addition, reinforcement or connection details of the in-situ bridge may not comply with modern standards and code requirements. Older reinforced concrete columns may not have sufficient confinement to prevent loss of core concrete during extreme seismic events.

Seven basic approaches outlined in Buckle et al. (2006) that may be considered during seismic retrofit of a bridge whose foundation is being reused:

- **Strengthening:** increases capacity of overloaded elements to resist greater forces or moments. This can refer to improving the elastic range strength or ultimate strength, depending on the type of bridge and seismic hazard spectra.
- **Improvement of Displacement Capacity:** Provides additional displacement capacity, usually by increasing seat width or allowing greater inelastic response from columns before failure.
- **Force Limitation:** Provides deliberate yield points to prevent adjacent members from becoming overloaded. This can include seismic isolation bearings or structural fuses.
- **Response Modification:** Fundamentally alters the manner that forces are transmitted through structure by providing additional stiffness to some members or modifying load path. May impact normal bridge behavior.
- **Site Remediation by Ground Improvement:** Ground improvement techniques can be employed to limit the potential for liquefaction, reduce site amplification, or prevent other hazards.
- **Acceptance of Control of Damage to Specific Components:** Allow member to be damaged during design earthquake, so long as it does not impact the stability of the structure, lead to collapse, or impact life safety. Crucial members of critical bridges are typically designed to remain in the elastic range during an earthquake.
- **Partial Replacement:** Replace bridge components that are unsuitable for reuse.

Superstructure Changes and Dead Load Reduction

Many times, superstructure changes can be made that limit the impacts on the reused substructures. One of the most common issues with substructures that can be addressed through superstructure changes is the runoff of contaminated water from the deck onto the bridge piers. This provides a path of chloride ingress into piers and often leads to corrosion. New expansion joint details and changes to the superstructure can prevent the runoff from reaching the substructure elements.

Another change that can be made to the superstructure is the use of lightweight concrete. The original superstructures for many existing bridges may have been constructed using normal weight concrete. The usage of lightweight concrete and other high-performance materials (stronger concrete, stronger steel, etc.) can lower the total weight of the bridge deck, often by 10 percent or more. Dead load from the superstructure can also be reduced by using a different form of the superstructure. Newer deck technologies, like Exodermic® decks (BGFMA 2017) can allow for lighter superstructures that decrease the total loading on substructures.

One technique that has been explored is the use of composite FRP decks to reduce the weight of the superstructure (Gangarao et al. 2007; Whipp 2001). The FRP deck can act compositely with the original steel girders, replacing much of the strength of the concrete deck with a fraction of the mass. The reduction in mass can allow for drastically reduced loading on the substructure elements or to eliminate the need for entire piers. An example of this approach is the Market Street Bridge in West Virginia, as documented by Gangarao et al. (2007). This bridge was a two-span continuous steel girder bridge with a composite concrete deck. The bridge was reconstructed with a honeycomb shaped fiber reinforced polymer (FRP) deck (figure 63) that was attached to the steel girders to allow for composite action. The FRP deck was able to replace some of the compressive contributions of the original concrete, but with a drastically lower mass. The existing center pier was abandoned in place, as the reduced dead loading allowed the two spans to be replaced as a single span. The existing abutments were also abandoned in place and new piles were driven behind the original abutments to allow for installation of new integral abutments. While not a reuse case, this system allowed for a variety of substructure alignments that would have not been possible with concrete deck.

Figure 63. Photo. Honeycomb FRP deck used to replace concrete deck

ADDITION OF GEOTECHNICAL ELEMENTS

General Considerations

When new foundation elements are to be installed adjacent to an existing and in use foundation, important considerations are: the capacity of the new elements, the capacity of the existing elements, constructability of the selected strengthening technique, connection of the new element with the existing foundation, behavior of the new foundation under various loading scenarios, and expected life of the strengthened foundation.

Soil Displacement

The installation of foundation elements can cause disturbance to the surrounding ground, reducing the capacity and performance of existing elements. Displacement methods of installing new elements, like pile driving or some ground improvement techniques, can cause issues with ground heave and swell. Excavation methods, like drilled shafts or micropiles, can cause settlement issues if soils cave into the hole during the installation. When drilling next to an adjacent structure, it is important to identify and take necessary precautions to prevent cave-ins. Understanding the contractor's preferred installation methods is crucial, as some methods will be inherently more prone to soil displacement or settlement.

Ground Vibrations

The vibration levels caused by various installation techniques can damage structures within the zone of influence of the installation. Building codes typically prescribe acceptable vibration threshold in the form of Peak Particle Velocity (PPV, in/sec), which limit the amount of vibrations that surrounding structures can be subjected to. Activities, such as pile driving, can cause distress in existing structures because of shock type excessive vibrations.

Site Clearance and Overhead Room

Some technologies are more suited to sites with low overhead room or site mobility than others. Technologies like driven piles or continuous flight auger (CFA) piles can be very difficult or impossible to install without adequate overhead clearance. Other technologies like drilled shafts can be installed without much overhead room, but at lower production rates. Technologies like driven piles and drilled shafts often use very large equipment, which cannot be easily maneuvered on sites with little room. Various technologies like micropiles, jacked piles, tiebacks, and helical piles can be installed with specialty equipment designed for low clearance spaces. An evaluation of the worksite can help determine which technologies are possible, and what limitations are imposed by the site and site conditions.

Environmental Considerations

Foundation strengthening technologies like drilled shafts, CFA piles, micropiles, and tiebacks can produce spoils during their installation. Sometimes, these technologies can be applied without producing significant spoil runoff from the site, but this may require additional equipment or slow production rates. Disposal of the collected soils can incur additional costs, especially when soils are contaminated. Performing strengthening work in river crossings may complicate some of the strengthening procedures if spoils or other solids are not allowed to be released into the river.

Loading Behavior

Understanding the loading characteristics of the combined system allows the foundation to be designed such that the full capacity of new and old elements is utilized at failure. This will maximize the effectiveness of both new and existing elements, although in practice this is not always feasible. If elements of the foundation are of vastly differing stiffness, then some elements can become heavily loaded and can reach their capacity before other elements experience sizable loading. This is not always an overriding concern, since many systems are inherently ductile. For instance, piles experiencing bearing capacity failure can undergo large deformations prior to actual damage. This behavior can help the system redistribute the load to other piles which have not yet reached their capacity. When other mechanisms of failure occur, such as failure of pile elements in compression, the system may not have the opportunity to redistribute load before an element loses all load-carrying ability. When non-ductile failure modes are present, differences between existing and replacement structures can become very important considerations.

Micropiles

Micropiles generally refer to small diameter (6 inches to 9 inches, 152 mm to 229 mm) piles which are installed through jacking, driving or by drilling and casting the pile in place (Aktan and Attanayake 2013). Drilled, cast-in-place micropiles are the most common type of micropiles. Because of this, the definition of micropiles is sometimes limited to drilled cast-in-place elements (Bruce and Juran 1997). Micropiles obtain their capacity from both end bearing and side resistance, however end bearing is typically neglected and only side resistance is relied upon (Sabatini et al. 2005). The end bearing contribution is generally regarded as negligible, since micropiles can attain very high grout-to-ground bond strength, have significantly greater area available for skin friction than end bearing, and the skin friction is mobilized from significantly smaller movements than required to mobilize end bearing resistance (Armour et al. 2000).

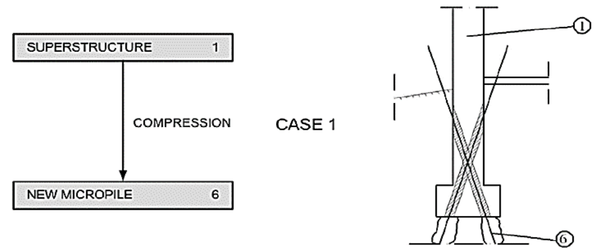
Since the reinforcement used in micropiles typically consists of a single bar placed in the center of a grout column, micropiles do not typically have large lateral capacities. The lateral capacity can be increased by using a permanent drill casing which can be abandoned in the drill hole. Specialty equipment allows micropiles to be installed in small areas with limited overhead. Drilled micropiles generally cause very little soil displacement and ground vibration, allowing them to be installed very close to existing load bearing foundation elements. Soil movement and ground heave can be easily avoided with the appropriate micropile installation techniques. Table 35 presents a breakdown of the advantages and limitations of micropiles.

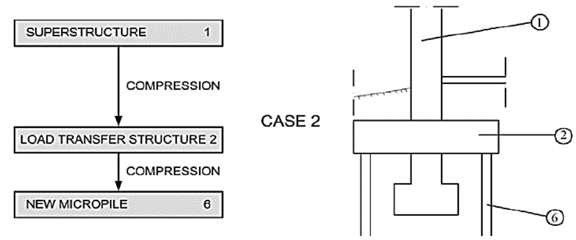
Table 35. Advantages and limitations for micropiles (after Aktan and Attanayake 2013)

Advantages	Limitations
The equipment is relatively small and can be mobilized in restrictive areas.	Vertical micropiles are limited in lateral load capacity.
Can be installed in all ground conditions.	More expensive than other options.
Cause minimal disturbance to adjacent structures.	-
Cause minimal noise and vibration.	-
Can be used in low head room conditions (6 ft minimum).	-
Can be used for underpinning existing foundations.	-
Can be installed as batter piles.	-

There are a variety of methods for connecting new elements to an existing foundation. Often, new elements are installed slightly outside of the footprint of the existing foundation and a transfer structure is installed to transfer loads from the existing structure to the new element. One notable exemption is when micropiles are drilled directly through exiting elements to enable connection without the need for a transfer structure. Jacking can be employed to ensure that the new elements take up the load without much additional deflection. Jacking is a commonly used technique for either arresting settlement or recovering previous settlement.

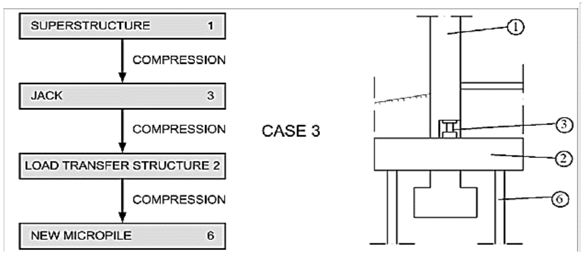
Lehtonen et al. (2010) have presented extensive discussion on available options for connecting existing superstructures to new installations of micropiles. Specifically, Lehtonen et al. (2010) has presented seven cases of pile connections to existing structures as illustrated in figure 64. Five of the cases involve jacking against the original structure, while the other two do not use jacking. Six out of seven options use a transfer structure to transfer the load from the piles to the original structure.

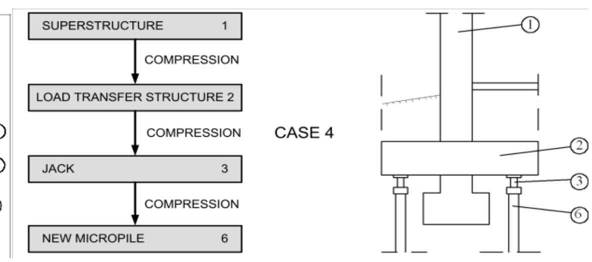




A) Case 1

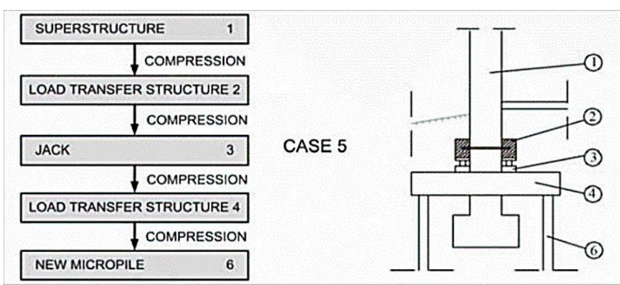
B) Case 2

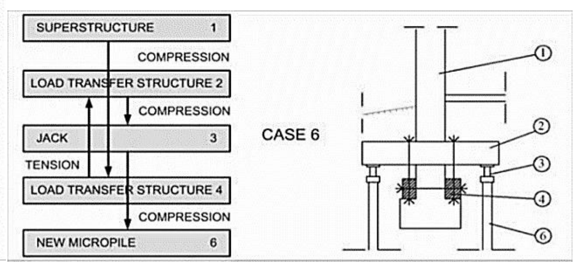




C) Case 3

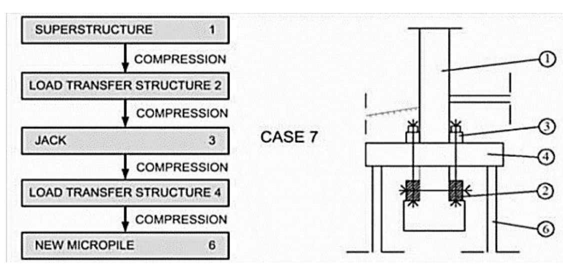
D) Case 3





E) Case 5

F) Case 6



G) Case 7

Figure 64. Illustration. Connection details for various cases

Jacked Piles

The jacking installation technique consists of pushing a steel tube through soft soil layers to the design depth or until the required resistance is achieved (Aktan and Attanayake 2013). Open ended or close-ended steel tubes can be employed, where the close-ended tubes can be filled with grout for additional axial stiffness and strength after installation. To jack the piles into the ground, a reaction force is required. Typically, if an existing structure is to be supported, this existing structure can be used as the reaction mass. The ultimate capacity of this installation is generally limited to the amount of force which can be applied during installation, implying there needs to be sufficient dead weight above these piles to facilitate installation.

This method produces a small to medium amount of soil displacement, with the close-ended piles producing a greater amount of soil displacement. Jacked micropiles do not generally produce significant amount of soil vibrations. Installation can be impeded by obstructions such as boulders, old foundations and debris. Since the primary equipment used for installation is a hydraulic jack, very little headroom is required for installation.

Driven Piles

Driven piles can be circular or square and are made from timber, steel, or prestressed concrete (Hannigan et al. 2006, Hannigan et al. 2016). Timber piles are light and inexpensive, making them well suited for projects where relatively shallow friction piles are installed. Steel piles are heavier and can resist higher loads, making them suitable for end bearing or combined side-friction end bearing piles. Since the cross sections of steel piles are generally smaller than those of timber and concrete piles, they generally cause less soil displacement during driving. Concrete piles generally have the highest capacities and are suitable when high resistances are required from each pile. They also cause the largest level of vibration and soil displacement and are unlikely to be the most suitable option for installation next to existing structures.

Installation of driven piles can cause high amounts of ground vibration or soil displacement, however some of this can be mitigated through proper pile selection or mitigating action such as pre-auguring. Since piles are constructed before they are installed, they can be fully inspected prior to driving, although damage can occur during driving, especially if driven improperly. Driven piles can generally be installed in any ground water conditions without impediment. However, overhead clearance can be a constraint when an impact hammer is used as a method of installation. Aktan and Attanayake (2013) have provided an overview of the advantages and limitations to using driven piles for foundation strengthening, as shown in table 36.

Table 36. Advantages and limitations for driven piles (Aktan and Attanayake 2013)

Advantages	Limitations
Pile can be inspected before it is driven into the ground	Driving activity might exceed both noise and vibration limits
Construction procedure is unaffected by ground water table	Low headroom is a constraint
Pile can be spliced and driven into deeper strata	Pile size is limited
Driven pile foundations are generally less expensive than drilled shafts	-

All forms of driven piles have the potential of producing ground vibrations that impact adjacent structures. The most important factors to consider when estimating the magnitude of ground vibrations are the pile size, pile type, hammer energy, and site soil conditions. Larger piles displace more soil and generally require more hammer energy, leading to higher amounts of energy transferred into the surrounding ground, while piles such precast concrete or shoed pipe piles will generally produce higher vibrations than shoeless pipe piles and H-piles. Driving through hard material or boulders will require larger hammer energies and will produce much larger ground vibrations than driving through soft materials. Driving through sands creates a volume of densified material at the pile tip more effectively than in clays, meaning driving through sand is generally more difficult than clays and produces higher vibrations.

The installation of driven piles will displace an amount of soil equal to the pile cross-section, which may be substantial for concrete sections or sections with closed ends. H-piles and open-ended pipe piles will displace less soil (assuming there is no formation of a soil plug during driving), although these methods can still cause heave in surrounding areas. When piles are driven into the bearing strata next to existing piles, pile heave can alter the load-displacement behavior of the existing piles. Monitoring existing piles during installation can help determine when any pile heave has occurred. When driven piles are installed adjacent to a shallow footing, soil heave is a much larger concern. Not only is there a possibility of foundation movement, but the soil displacement can lower the capacity of the footing and cause various stresses to build up in the foundation and superstructure.

Drilled Shafts

The design and installation of drilled shafts is discussed in detail in the FHWA manual, GEC-10 (Brown et al. 2010). Drilled shaft foundations are created by boring a hole of several feet in diameter and casting a reinforced concrete shaft in place. The concrete shaft can resist axial and lateral loads through a combination of skin friction and bearing. Drilled shafts generally range from 3 ft to 12 ft in diameter and can be installed up to 200 ft to 300 ft below ground surface (Brown et al. 2010). Due to their very large diameter, drilled shafts can often resist large lateral loads, unlike slenderer micropiles and driven piles. Due to their large cross section and high axial force capacity, drilled shafts are economical only when there is a competent bearing stratum such as sound rock or very hard soil within 100 ft to 200 ft of the ground surface. While drilled shafts can be used in a wide range of soil profiles, the cost of installation is highly sensitive to site conditions and installation technique. They can be installed in any ground water conditions. However, high groundwater can be a complicating factor that increases the cost of installation.

Drilled shafts do not generally create the level of ground vibrations caused by driven piles. They generally do not displace large amounts of soil that could harm adjacent foundations and structures. While common drilling equipment is quite large, low headroom options are available, although production rates will generally suffer when drilled shafts are installed in low overhead and limited space situations. Since drilled shafts can have large lateral and axial capacities, it is often economical to install a single large element rather than a group of micropiles or driven piles, ultimately saving space and eliminating the need for a costly pile cap. Table 37 shows advantages and limitations of drilled shafts (Aktan and Attanayake 2013).

Drilled shafts produce almost no ground vibrations in comparison to driven piles and can often be used adjacent to existing structures. Since drilled shaft installation is generally performed with large machine, vibrations and noise are generated during the process, which can be a nuisance to nearby businesses and residences. However, these vibrations do not cause distress to nearby structures. Drilled shafts can be installed with virtually no soil displacement or ground movements when the proper precautions are taken. When dry excavations are made before casing installation, the hole is generally slightly oversized to facilitate easy installation into the excavated hole.

Table 37. Advantages and limitations of drilled shafts

Advantages	Limitations
Suitable for a wide range of ground conditions	Requires an experienced and capable contractor; usually a specialty subcontractor
Suitable for large axial as well as lateral loads	Batter piles are not possible
Single shaft can be used without a pile cap when space is constrained due to existing structures or foundations	May not be efficient in deep soft soils without a suitable bearing foundation
Low noise and vibration levels	Might require specialized equipment for special installations
Can be constructed with limited headroom (25ft or less)	Construction is sensitive to groundwater or challenging drilling conditions
-	Construction quality control and quality assurance is a challenge
-	It is challenging and costly to repair defects
-	Longer Construction time than other deep foundations

Continuous Flight Auger (CFA) Piles

The design and installation of continuous flight auger piles is discussed in detail in the FHWA manual, GEC-8 (Brown et al. 2007). Unlike conventional drilling methods, which have an auger attached to the end of a drill string, CFAs have one continuous hollow stem auger that makes up the entire drill string. The major advantage of this approach is the ability to drill and install piles without the need to continually remove the drill string and auger from the ground. This allows holes to be drilled in cohesionless and soft soils without using a casing to support the surrounding ground. These factors allow much more rapid pile installation than more conventional techniques, assuming the site conditions are suitable for this technique.

The basic process of installing a pile using a hollow stem auger is to advance the auger to the design depth while rotating the auger to remove spoils to the ground surface. When the drill string reaches the design depth, concrete or a sand/cement mixture can be pumped down the center of the hollow string auger as the auger is withdrawn. Generally, the auger is still rotated "downward" during removal, so that spoils continue to be brought to the ground surface. After removal of the drill string and placement of concrete, a rebar cage can be inserted; however, this is a time-sensitive procedure as the cage needs to be inserted before the concrete begins to cure.

Headroom restrictions are a serious concern when using this installation technique. Ideally, CFAs are installed with single string augers, so the amount of headroom required would be at least much as the depth of the hole to be drilled. There are some low-headroom techniques for CFA installation. However, because the entire drill string needs to be removed so that the rebar cage can be installed before the concrete has a chance to cure, the low-headroom techniques are fairly limited in application. Advantages and limitations of CFA piles are shown in table 38 below (Aktan and Attanayake 2013).

Table 38. Advantages and limitations of CFA piles (Aktan and Attanayake 2013)

Advantages	Limitations
Rapid Installation	Not suited for soils with rocks and boulders
Limited noise and vibration	Groundwater should be very deep
Possible, but challenging to install under low headroom	Specialized equipment is required
Applicable in weak soils	Procedures have not been fully developed
-	Need containment systems to control debris and grout spills
-	Drilling may reduce the confinement of the neighboring piles
-	Construction quality control and quality assurance is a challenge
-	It is challenging and costly to repair

Generally, CFA piles are installed using an auger in a single piece. Augers can have additional pieces added to it as it is drilled into the ground, but this greatly complicates the installation procedure. Having the auger being a single flight means that the required overhead clearance needs to be at least as deep as the hole being installed to avoid using multiple sections. Withdrawal of the auger becomes significantly costlier when multiple sections are used, increasing the chance of cave-ins and soil collapse.

CFA piles do not typically induce excessive vibrations in the surrounding ground and adjacent structures. When trying to install CFA piles in ground with large boulders, vibrations can occur as the auger scrapes against the boulder, though this concern is generally lower than concerns about soil displacement. Soil displacement and overmining is a prominent concern for CFA piles, especially when installed near an existing foundation. The risk of cave-in is not a particularly large concern while the auger is in the ground, it becomes time-sensitive to install the rebar cage and concrete before the hole collapses after removal of the auger. Hole collapse under or directly next to an existing foundation could cause serious damage to the existing foundation.

Tiebacks and Soil Nails

Tiebacks can be anchored into rock or competent soil and provide lateral restraint to retaining walls and structures (Sabatini et al. 1999). Tiebacks are reinforced using either threaded bars like micropiles, or by using steel tendons capable of providing tensile resistance. They are often prestressed to limit the amount of wall movement required before the anchor resistance is mobilized. While tiebacks are commonly used for temporary earth support, there are commercially available options for permanent earth support (design life of 100 years). These systems typically have multiple layers of steel protection, including grout, lubricated sheathing on the strands, and corrugated plastic outside of the grout. The lubricated sheathing allows the prestressing forces to only be transferred to the "bond zone," ensuring that resistance is being provided by soil outside of the failure plane. The design and installation of tiebacks and ground anchors are covered in greater depth in geotechnical engineering circular (GEC) 4: *Ground Anchors and Anchored Systems* (Sabatini et al. 1999).

Soil nails are generally smaller than tiebacks and can also be anchored in soil or rock. Soil nails are usually installed in a denser grid than tiebacks and work to reinforce the retaining wall by anchoring the wall face to a larger wedge of soil or rock (Lazart et al. 2015). Acting as a system, the wall and soil provide greater stability and resistance to sliding and overturning than the wall or rock/soil face alone would provide. Unlike ground anchors, soil nails by definition are not post-tensioned. Soil nails are discussed in greater depth by GEC 7: *Soil Nail Walls* (Lazarte et al. 2015).

Conventional Underpinning & Footings

Shallow footings for underpinning are typically connected to the structure above directly through dry-pack, low slump cement. The most frequent use for conventional underpinning is for shallow foundations that are not deep enough or are likely to be undercut by construction activities. The underpinning is typically done in small sections to avoid undermining the foundation during construction. This method requires maintaining a dry work area in the underpinning pit during construction, complicating the use of underpinning near rivers and other bodies of water.

GROUND IMPROVEMENT

Global Ground Modification

Global ground modification refers to techniques designed to enhance the suitability of the ground near the foundations. Available technologies for global ground modification include compaction grouting, jet grouting, permeation grouting, soil mixing, stone columns, and dynamic compaction. An in-depth overview of these technologies and others less applicable to reuse is provided in GEC 13 (Schaefer et al. 2016). Ground modification may be an attractive solution during reuse, as various technologies can be employed to: reduce liquefaction potential, improve bearing capacity, stabilize abutments, and stabilize slopes. Table 39 provides a list of some available technologies, their purpose, and a description of the technology.

Table 39. Potential ground improvement technologies during foundation reuse

Technique	Uses	Description
Compaction Grouting	• Liquefaction mitigation • Bearing Capacity • Improve passive resistance • Settlement Reduction	Densifies soil by injecting grout volume that displaces original soil. Most effective in sandy soils, can cause ground heave and soil displacement near injection
Jet Grouting	• Liquefaction mitigation • Bearing Capacity • Settlement Reduction	Soil is jetted out of ground in columns and replaced with grout mixture. Can lead to ground displacement especially when elements are loaded during installation. Can be used on sandy and fine-grained soils
Permeation Grouting	• Liquefaction mitigation • Bearing Capacity • Settlement Reduction	Grout is injected into soil and allowed to permeate into the void space of soil to improve soil performance. Most effective on cohesionless soils
Soil Mixing	• Liquefaction mitigation • Bearing Capacity • Settlement Reduction • Improve passive resistance • Reduce active pressure	Rotary tool advanced as grout injected into ground. Results in a column of mixed soil and grout with improved properties
Stone Columns	• Liquefaction mitigation • Bearing capacity • Soil densification • Settlement Reduction	Provides some lateral stability to soil and some compaction during installation. Can allow for excess pore pressures to dissipate. Some methods of installation densify the surrounding soil to increase stiffness and reduce settlements.
Dynamic Compaction	• Liquefaction mitigation • Bearing Capacity • Settlement Reduction	Densifies soils by repeatedly dropping large weight from crane. Improves soil properties where applied.

Pile Improvement Techniques

Ground modification performed near driven piles can improve the shaft (side) resistance of the pile or the end bearing capacity. Technologies that increase the stiffness of the soil surrounding piles will also increase the lateral capacity and pile performance in response to lateral loads. Potential technologies include jet grouting, shaft grouting, permeation grouting, compaction grouting, and base grouting. These technologies show large promise but are somewhat unproven or proprietary. Testing is often performed to confirm additional capacity. Scour countermeasures can be employed to stop soil erosion around driven piles that are susceptible to scour.

Replacement of Backfill

In some cases, it may be possible to replace backfill with lightweight fill, commonly in the form of extruded polystyrene foam (EPF) blocks. The design and implementation of this technology is discussed in NCHRP Report No. 529 (Stark et al. 2004). Since polystyrene blocks are much lighter in comparison with the soil being replaced, the lateral loads on the abutment can be reduced drastically. These blocks range in density from 0.7 lbs/ft^3 (11.2 kg/m^3) to 3 lbs/ft^3 (48 kg/m^3) and have a compressive strength of 317psf (15 kPa) to 5850 psf (280 kPa), according to ASTM Standard D6817 (2017). Geofoam backfill was used during the replacement of the

Jackson Road Bridge over Rout 2 in Lancaster, MA (GTR 2014). The use of lightweight backfill reduced the eccentric loading on the timber piles, allowing for their reuse.

Scour Countermeasures

When existing scour and/or future predicted scour at a bridge foundation is significant, technologies to eliminate existing scour and/or mitigate future hydraulic risks may be employed. Scour countermeasures are technologies employed to mitigate the potential or magnitude of scour expected during a flood event. Technologies available for bridge piers and abutments typically involve hydraulic countermeasures, riverbed armoring, or structural changes (Lagasse et al. 2009; Agrawal et al. 2007). Hydraulic countermeasures deflect, channel, or control the flow of water as to not produce scour at the bridge pier, even under flood conditions. Riverbed armoring prevents the soil from scouring even when exposed to flood waters. Structural countermeasures can be used to strengthen the foundation to withstand the projected scour or to modify the geometry to reduce the expected local scour around piers. HEC-23 (Lagasse et al. 2009) also discusses biotechnical countermeasures such as vegetated riprap, woody mats, or root wads. These technologies are considered “soft” protection and rely on living plat material to prevent erosion. These technologies are generally unsuitable when failure of the countermeasure could lead to bridge failure. Monitoring using fixed or portable instrumentation or visual observation is discussed as a technique to manage bridges vulnerable to scour, although this approach does not mitigate the scour risks in line with the requirements of modern codes.

The Handbook of Scour Countermeasure Design (Agrawal et al. 2007) discusses various possible structural countermeasures that can be employed to prevent streambed erosion, as shown in table 40. In addition to the listed approaches, various proprietary systems are available to reduce potential scour at piers and abutments. These systems are often hydraulic/armoring countermeasures that function by diverting flow around piers, generating vortices that counter traditional scour vortices, or armoring the ground surface. These countermeasures can be highly specific to the type of piers/abutments present, as well as river and soil characteristics.

Table 40. Possible scour countermeasures (Agrawal et al. 2007)

Countermeasure Type	Type of Scour available for	Description	Advantages	Disadvantages
Vegetative planting, grasses, trees, shrubs	Degradation, lateral erosion	Grasses, trees, and vegetation planted to counter scour	Low cost, natural appearance	Difficult to plant on steep banks or with big stones
Packed and compacted riprap on geotextile	Local scour, degradation, lateral erosion	Graded broken rock placed below riverbed and covered with soil	Relatively low cost, minimal maintenance, easy to construct	May need hard to find oversize stones, disturbance to channel ecosystem, labor intensive
Artificial riprap	Local scour, degradation, lateral erosion	Man-made riprap including tetrapods/toskanes	Useful when large riprap is not available, can be precast at low cost, no impact to water quality	May prevent vegetative growth, more expensive than natural riprap
Gabions, Reno mattress on geotextile	Local scour, degradation, lateral erosion	Wire mesh baskets filled with loose stones	Rocks inside baskets do not move, can be used with steep sloped banks, does not require as large stone as riprap alone	Wire may break due to corrosion or vandalism, debris may get trapped, requires regular maintenance
Precast concrete interlocking blocks	Local scour, degradation, lateral erosion	Cellular concrete blocks placed as protection	Readily available, easier to find than some large riprap	Prevents vegetative growth, Liable to move in floods when not anchored
Cable-tied blocks	Local scour, degradation, lateral erosion	Concrete blocks interconnected with steel cables	Minimum maintenance, will not wash out in floods	Not suitable for pile bent bridges, requires divers for installation, steel cables likely to corrode
Sacked concrete, grout filled bags	Local scour, degradation, lateral erosion	Fabric bags filled with concrete and stacked to provide protective layer	Suitable for sandy soils, useful for filling scour holes	Prevents vegetative growth, possible cement washout, toe may still undermine

DECISION MAKING TOOLS

"GeoTechTools" is a website developed by the SHRP 2 R02 research team at Iowa State University. The website provides solutions and technologies available to counteract common problems related to slope stability, foundation capacity, and other geotechnical issues. The catalogue of techniques contains fact sheets, photos, case histories, design guidance, QC/QA procedures, cost estimations, and specifications. Users can upload their own case histories to add to the available information.

CASE STUDIES

Several case studies where foundation reuse involved foundation strengthening are discussed in this section to highlight key aspects of strengthening.

Milton Madison Bridge, between Madison, IN and Milton, KY

Overview of Issues

In all, five piers (existing Piers 5 through 9) were investigated for potential reuse. Piers 5, 6, 7, and 8 were located within the river, while Pier 9 was located within the flood plain, but not within the long-term river banks. Piers 5 and 9 were exposed to deicing runoff due to joints in the original deck system and showed significant chloride intrusion and resulting deterioration. Piers 6 through 8 had not been exposed to the same level of chloride, though each had experienced at least some previous spalling and repair. Cold joints from the original construction were observed, and these areas experienced additional deterioration. Carbonation and accelerated erosion were noted on the exterior surface around these joints. The erosion had exposed the reinforcement cage, which showed considerable amounts of corrosion. The yield strength of the original reinforcement was not known with confidence, although the layout was corroborated with GPR surveys.

The primary structural capacity related concerns with the existing foundations were the moment capacity of the pier stems and caisson. The moment capacity of the pier stems and caissons was limited by the lack of reliable reinforcement in the existing structure. When analyzed as an unreinforced concrete, the structure did not have sufficient capacity. Use of lightweight concrete on the deck was not evaluated due to life cycle cost concerns and because the loss of deck dead weight may have increased the amount of tension in the pier stems during bending. The overturning capacity of foundation was deemed to be sufficient only if scour of the riverine piers was prevented. Due to the amount of potential local scour, soil improvements were not available to reduce the tensile stresses on the piers through additional lateral restraint.

Service Life Extension Requirements

Two primary options were considered for repair of chloride affected areas: Electrochemical Chloride Extraction (ECE) and replacement of existing chloride impacted concrete. Existing Piers 5 and 9 would have required extensive cover replacement, while the chloride content in remaining piers had not yet reached the threshold. Placement of a minimum 3” (76 mm) of High Performance Concrete (HPC) additional cover was deemed necessary to provide the recommended 75-year service life for all piers.

Structural Repair

It was determined that Pier 5 could be eliminated entirely by increasing the span length in this portion of the bridge. This increased the loading on the remaining piers but eliminated the need for any rehabilitation of Pier 5. The remaining pier stems were encased in a 24 inches (0.61 m) thick layer of HPC. The new concrete was tied into the existing concrete using hooked dowels that were epoxied into the original concrete face. A completely new layer of epoxy coated rebar was used to provide longitudinal and transverse reinforcement to the new pier. For Pier 9, the chloride impacted concrete cover was removed prior to be being encased in HPC. Since the concrete was encased prior to removal of the original deck, the encasement was assumed to go into tension during removal and return to a zero-stress state after the original dead load was replaced. The additional dead load from the heavier deck was expected to be carried between a combination of the new concrete and the original concrete. Regardless, the reinforced encasement section was designed to carry the entirety of the loading on the bridge. Although a conservative assumption, it did not impact the design as the 24 inches of encasement was deemed necessary to provide sufficient service life. The concrete encasement was modeled as a hollow reinforced concrete section using spColumn (StructurePoint LLC 2016). The wider spacing of the new superstructures bearings generated high horizontal tensile stresses between the two bearings. Post-tensioning was used to overcome this tensile stress and keep the top of the pier stem in compression. Strut and tie models were used to estimate the capacity of the top of the pier stems.

The caissons were cored with 4 inches (100 mm) diameter and 4-5/8 inches diameter holes (figure 65), and bundled #11 or bundled #14 bars were placed down the holes and grouted into place to act as tensile reinforcement (figure 66). The bars were extended up into the pier stem reinforcement to provide adequate load transfer capacity between the caisson and the pier stem. The caissons were analyzed as reinforced concrete columns using spColumn.

Figure 65. Photo. Coring of caisson concrete from inside of cofferdam

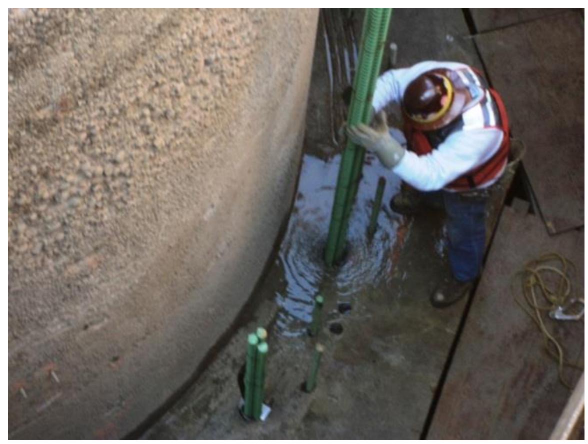


Figure 66. Photo. Insertion of reinforcement into cored holes

Ground Improvement/Scour Countermeasures

All four reused piers were protected from future scour using rip rap large enough to resist scour, as determined from the Isbach Equation from HEC-23 (Lagasse et al. 2009). The three riverine piers required 15 ft (4.6 m) (12 ft of local scour plus 3 ft of contraction scour) of rip rap with a D_{50} of 18 inches (45.7 cm) and a D_{15} of 11 inches (28 cm). The rip rap was placed on a filter fabric extending two times the pier width in all directions from the pier. The existing Pier 9, which was on land but in the flood plain, required slightly smaller riprap placed to a depth of 45 inches (1.14 m) below the existing soil grade.

Torrey Pines Bridge, Del Mar, CA

Deteriorated Column Repair

The column repair was primarily intended to address the ongoing corrosion caused by chloride ingress. The first step was removal of damaged, spalled, and delaminated concrete up to 1 inch (25 mm) below the inner reinforcement layer. The cover concrete was also removed in areas where seismic shear retrofit was required (discussed below). An impressed current cathodic protection system with titanium mixed metal oxide anodes was installed in the sections where the concrete cover was removed. The cover concrete was then replaced by placing new formwork around the columns and pouring new concrete. The cap beams, struts, and columns underneath the shear retrofit were patched where necessary, and discrete galvanic anodes were placed to inhibit further corrosion.

Seismic Shear Reinforcement

To improve both the shear capacity of the pier columns, a shear reinforcement retrofit was performed. Additional shear reinforcement was added to the upper portions of the all pier columns after removal of the concrete cover. This retrofit increased the shear capacity of the columns beyond the demand required from the seismic analysis. The new shear reinforcement consisted of continuous ties that were assembled in place through use of mechanical couplers.

Bearing Replacement

The bearings were replaced at all bents to allow for better properties during seismic events. Spherical PTFE bearings were used at all bents one of the abutments (with the other being integral), although with varying properties. The skewed bents and the two non-skewed bents on either side of the skewed bents could translate freely in the longitudinal and transverse directions, except for a longitudinal fixity. Abutment 1 and bents 2 and 3 were allowed to be free longitudinally and were pinned transversely. Bents 8 through 12 were designed to be pinned-pinned. These design types were realized by selecting the appropriate PTFE bearings for the intended purpose.

Abutment Strengthening

Both abutments needed replacement due to their inability to restrain the deck during seismic excitation. To preserve the historic nature of the bridge, the original abutments were left in place and minimally repaired, while the new abutments were hidden behind the existing abutments. The new abutments were founded on 5 and 6-foot diameter drilled shafts that allowed for large lateral restraint during ground motions. One of the new abutments was made to be integral with the new superstructure, and the other supported the superstructure with PTFE bearings. Placement of the new abutments behind the existing ones slightly increased the overall length of the bridge.

Skew Bent Strengthening

The main improvement to the skew bents came from their seismic isolation that drastically reduced the forces applied at the tops of columns. The crash walls along the sides of the bent, however, caused significant shear loading in the weak direction that needed to be transferred to the abutment. To transfer this loading, the shear wall was extended down to the tops of the foundation elements.

Ground Improvement

Compaction grouting was performed in areas of potential liquefaction to prevent both liquefaction and slope failure due to liquefaction. SPT and CPT soundings were performed before and after compaction grouting was performed. These tests showed substantial improvements to the density of the previously liquefiable soils.

Georgia Street Bridge, San Diego, CA

Three alternatives for the stabilizing the abutment walls were considered: to use existing wall as a back form and pour a 9 inches encasing layer of concrete, to replace the entire wall by demolishing and replacing it section by section, or to drill behind the existing wall to install soldier piles and deconstructing from the top down. The first alternative was chosen, as it had the lowest cost and was easiest to implement. The new walls were attached to the existing deadman anchors, and a grid of soil nails was added to the wingwalls to provide adequate resistance. The abutment walls were laterally reinforced with soil anchors prior to installation of the new concrete. The existing concrete walls were left in place and not relied upon for capacity. New soil anchors were drilled through the anchor blows to increase lateral stiffness and capacity.

Following are the major components of retrofit Alternative 1, listed by Simon Wong Engineering (2015) and illustrated in figure 67:

- Superstructure abutment soil improvement
- Superstructure abutment retrofit for passive resistance.
- Shear wall placed between spandrel columns of Bent 1
- Shear retrofit of the arch-ribs along with an additional ¾ inches (19 mm) cover
- Replacement of the spandrel columns
- Uplift and transverse key guides for arch-rib to footing interface
- Deck slab and barrier replacement, remove excess AC
- Abutment and wing wall retrofit
- Thrust block stabilization

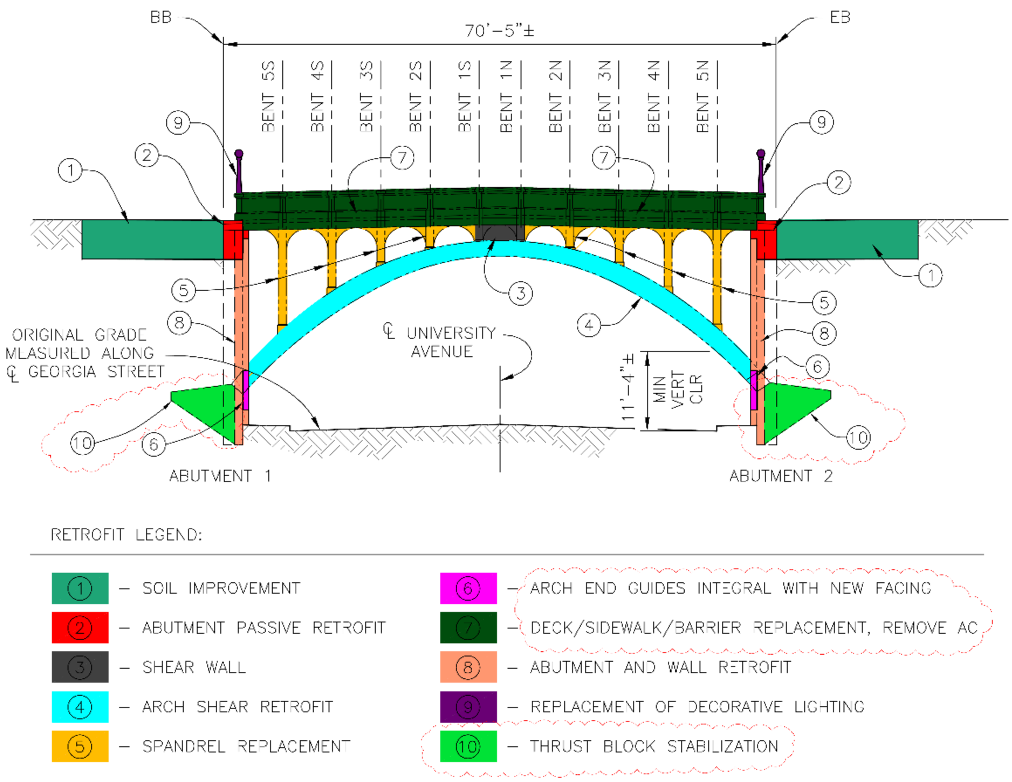


Figure 67. Illustration. Georgia Street Bridge rehabilitation alternative 1

Infill shear walls were placed between the spandrels connecting the arch rib to the deck to make up for inadequate shear strength of the spandrels. The spandrels themselves were completely replaced during reuse, as retrofit of the existing spandrels was considered impractical due their small size. The arch ribs had their cover concrete entirely removed through hydrodemolition and replaced with new cover concrete. Other alternatives that were considered were encasing the existing arch ribs in additional concrete, complete replacement of the arch ribs, and only minor patching and repair. Patching and encasement were expected to do a poor job of preserving the historical appearance of the arch ribs. Removal of the cover concrete also allowed for removal of carbonated concrete, allowing for adequate service life.

Huey. P Long Bridge, Jefferson Parish, LA

The original five riverine piers were reused, however substantially more capacity was required to support additional load from widening. The upper portion of the original piers had gothic arch comprising of two columns that extended from the lower pier to the bridge bearings. Since the new superstructure was substantially wider, these columns would not be adequate by themselves. Two main options were considered: encasing the upper portions of the piers and placing wider supports on this encased section or bypassing the original concrete columns. It was determined the most feasible option was to bypass the upper pier columns altogether with a new steel frame. The steel frame transfers the widened superstructure load to three points on the lower portions of the pier below the arch. The outside two bearings would bear on new encasement concrete, while the inner bearing would bear on the original concrete pier. The encasement and the original pier were deliberately left non-composite with each other to simplify the calculations of load transfer to the lower pier and allow for shrinkage of the newly placed concrete. The lower portion of the pier was also encased with concrete, though this concrete was made to be composite with the original section. Hooked rebar was doweled into the original piers and the new concrete poured around it. A new reinforcement cage was provided for the encasement concrete. The placement of the steel frames was done without requiring closure of the bridge traffic. New trusses were then attached to the steel frames prior to removal of the original truss, greatly shortening the amount of disruption to normal traffic. The new approach piers added outside of the river were founded on driven piles supporting hammerhead style piers.

NJ Route 72 Bay Bridges, Ocean County, NJ

Piles that had experienced excessive section loss were jacketed with a protective cover that prevented additional section loss and restored stability to the piles. Scoured areas were filled in using a combination of rip rap and grout filled bags. Countermeasures were installed to protect the stream bed near the piles from further scour.

Mississagi River Bridge, Ontario, Canada

Five alternatives were considered to retrofit the three piers founded on overburden material, as described by Li et al. (2014):

1. Replacing the entire bridge including substructures;
2. Regrading the river bottom and improving rock and scour protection at each pier;
3. Underpinning the existing foundations using concrete caissons outside the sheet pile cofferdams driven to bedrock and tying back to existing pier shafts;
4. Underpinning the existing foundations using concrete caissons outside the sheet pile cofferdams founded on bedrock and tying back to the existing pier shafts; and
5. Underpinning the existing foundation using micropiles installed inside the sheet pile cofferdams through to bedrock and tying back to existing pier shafts using new caps.

To minimize disruption to traffic, reduce the environmental impacts, and improve the constructability, alternative number five using micropiles was chosen. The micropiles were drilled directly from the bridge deck (see figure 68), by drilling through the deck concrete then through air until a ledge between the pier and cofferdam was reached. The micropiles were cased with permanent casing through the entire concrete footing and soil until 0.5m into bedrock. A total of twenty-four micropiles were installed at each pier. Figure 69 shows the micropiles being grouted during installation.

Figure 68. Photo. Micropile installation through bridge deck with casing down pier face

The micropiles were connected to the piers through a transfer structure that avoided the existing footings and transferred all foundation loads through the micropiles. The existing footings and sheet pile cofferdams were abandoned in place and the micropiles were directly connected to the lower portions of the piers. Twelve compressive struts made from steel rails were placed into each pier (figure 70). The struts were connected to the top of a micropile on each side of the pier and used to transfer the compressive loads into the micropiles. Threaded DYWIDAG bars were attached at the base of the pier to act as a longitudinal tie. Reinforced concrete was poured around the entire assembly to encase and protect the steel.

Figure 69. Photo. Grouting of micropiles

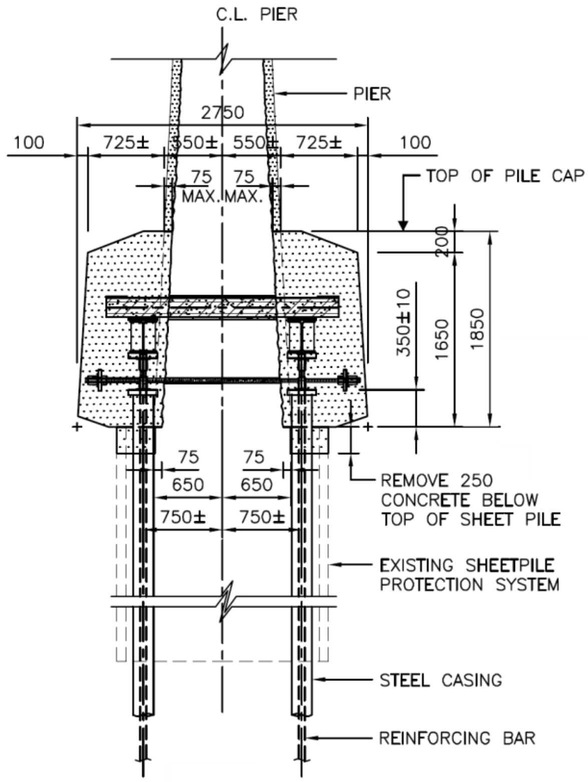


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Figure 70. Illustration. Transfer system to attached micropiles to piers

After installation of the micropiles, non-scourable rip rap was used to fill in the preexisting scour holes. The placement of non-scourable material was needed to ensure the micropiles had adequate lateral resistance. The existing pier caps above the transfer structure were encased with new reinforced concrete. The new caps featured extended ice nosing to reduce potential loads from ice flows. Expansion joints on the new bridge deck were positioned to reduce the amount of deicing runoff impacting the piers.

Henley Street Bridge, Knoxville, TN

Drilled shafts were added underneath the riverine piers and designed to take approximately 80 percent of the new design loads. The shafts were designed using data obtained from the soil and rock borings and sampling. The spandrels that connected the arches to the superstructure were replaced with new elements.

Jackson Road Bridge, Lancaster, MA

The Jackson Road Bridge did not require substantial strengthening or repair to the abutments, piers, or timber piles. However, the original analysis indicated that some abutment piles were overloaded, primarily due to the eccentric loading placed on them from the lateral loads caused by the abutment backfill. Reducing this lateral loading prevented the overload of these piles, so lightweight geofoam backfill was used to replace the soil backfill behind the abutments, reducing the lateral loading. This reduced the eccentricity of the loading on the piles, thereby decreasing the axial loading on the overloaded piles.

CHAPTER 8. DESIGN OF NEW FOUNDATIONS FOR REUSE

INTRODUCTION

The design and construction of a new bridge foundation can incorporate several considerations that reduce the complexity of future reuse or widening. Planning for reuse prior to initial construction can be advantageous in the following scenarios:

- Future addition of lanes and public transport options
- Future alignment changes, new ramps, overpasses, and adjacent construction
- Heavier vehicles and alternative uses
- Replacement of the deck without foundation replacement

Future addition of lanes or public transit options may require either that a stronger or wider foundation than necessary is initially built initially or that the foundation be strengthened mid-life to carry the additional load. Alignment changes may result from new road connections, underpasses, etc. that will require construction activities next to the existing element during its service life, potentially while in use. Planning for increasing vehicle weight or non-vehicular uses that alter the loading may require a stronger foundation than initially needed. The foundation can either be provided this additional strength up front or designed to accommodate the required strengthening. Future reuse can also be planned for as part of a long-term cost-reduction measure, by planning for foundations to have service life beyond the initial design life of the bridge.

DESIGN CONSIDERATIONS

Geometry

The spatial requirements for potential changes can be compared with the preferred layout of the new foundation. Details on the shape, bearing locations, and pile placement can often be changed to accommodate the future bridge alignment. The initial foundation can either be built to handle both geometries or provided with adequate space for additional components and their installation. Other details, like the location of footings, dolphin placement, scour countermeasures, and current alignment can be adjusted to facilitate potential reuse.

Substructure Selection

Selection of the above ground details of the foundation depends on the expected or design service life as well as possible extensions. Certain foundation types, like pile bents or steel lattice foundations can be adequate for normal service lives in the 50 to 75-year range but may have limited ability for an extended service life due to normal deterioration. Other technologies, like wall piers and large diameter columns, may present higher initial costs, but with lower maintenance costs and less deterioration during the initial service life. Details like concrete cover thickness, reinforcement type, and element size may be adjusted to accommodate better resistance against future deterioration. Consideration of potential strengthening option during initial construction can help reduce strengthening costs if required in the future.

Footings

Important considerations for bridges placed on shallow footings are the geometric and capacity requirements of potential reuse. Since a large portion of the costs associated with footing installation are from excavation, minor additions to the footing size may not add an appreciable amount of cost to the initial project. If these additions were not performed, however, future excavation and footing addition for strengthening could add substantial cost and complexity to the reuse design.

Deep Foundations

Pile Cap Placement

In terms of initial cost, it is often favorable to not install pile caps deeper than they need to be installed, as pile caps with a deeper bottom will require additional excavation and additional concrete. For service life considerations, however, it may be desirable to install pile caps slightly deeper than otherwise needed. If possible, ensuring that the long-term groundwater table is above the bottom of the pile cap will greatly extend the lifespan of the underlying piles, as submerged piles do not typically deteriorate due to low oxygen availability.

Pile Type and Service Life

The selection of pile type (e.g., concrete, steel, timber, composite, etc.) is generally based on initial total cost, which is impacted by the geometry of the foundation, subsurface conditions, site considerations (vibrations, soil displacement), and material availability. Piles that are in harsh environments can have additional costs related to service life impacts and maintenance. During alternative selection, considering potential service life beyond the initial design can influence the selection process. Driven piles also have specific considerations depending on the material type.

Steel Piles

Han et al. (2003) found that steel piles in soil can corrode at a rate of 1.33×10^{-3} in/year. Steel piles will typically have a design sacrificial thickness, expected to corrode during the initial service life, which is not relied upon for structural capacity. Selection of slightly larger piles will add to the initial material costs of the foundation but will allow for additional sacrificial thickness and an extended service life. Various alternatives (FRP wrapping, encasement, coal tar epoxy, etc.) are commercially available that can extend the service life of steel piles. These coatings might require future maintenance and replacement that increases their suitability for reuse.

Timber Piles

The service life of non-submerged timber piles is typically lower than that for other piles, despite timber piles having relatively low up-front cost. In some cases, although the final cost of using timber piles may be lower than other piling options, future reuse considerations may make durable options attractive.

Concrete Piles

Concrete driven piles are generally extremely corrosion resistant, unless exposed high levels of chlorides or other ions. Piles installed in areas of potential corrosion can be equipped with FRP wrapping, galvanic protection, or impressed current cathodic protection. For these to work, the prestressing steel and hoop reinforcement needs to be electrically contiguous throughout the section. Including these design details in the initial use of the pile will add to the up-front cost but will allow for greatly extended service lives.

Composite Piles

Fiber-reinforced polymer (FRP) composites for piles are gaining traction among both designers and contractors (Zyka and Mohajerani 2016, Guades et al. 2012). Composite piles have been used primarily for fender piles, waterfront barriers, and bearing piles for light structures (Juran and Komornik 2006). Better corrosion resistance, high-strength and low stiffness (relative to steel and concrete), and longer durability make FRP a cost-effective solution for piles. Therefore, using FRP pile can increase the service life span and consequently will increase the future reuse potential significantly.

Reserve Capacity for Future Reuse

Older bridges often need to be replaced because as traffic demands increase, the superstructure or the substructure goes in poor condition. Assuming observed trends continue, traffic loads will continue to increase. These trends mean that when considering replacement at the end of the initial service life, it may be desirable to construct a new superstructure on a foundation subjected to increased loading. Providing these new foundations with additional geotechnical capacity can allow for cheaper and more simplistic superstructure replacement and foundation reuse. An example of this scenario was observed with the Huey P. Long Bridge case study, which was found to have significantly more geotechnical capacity and concrete strength than the initial design intent. The bridge was widened, and the existing foundation resisted significantly increased loading without geotechnical improvements. While this reserve capacity was not explicitly planned for during initial construction, it provides an example of the potential cost savings when this reserve capacity is available.

Additional Test Piles

The decision to reuse a bridge foundation will depend on detailed assessments for integrity, durability and capacity. These assessments, particularly the confirmation of capacity through testing, can become difficult because of limited access to piles or drilled shafts. The accuracy, ease, efficiency, and speed of assessment of piles and shafts during potential reuse can be improved by incorporating test pile(s) beside the main load bearing piles. These test piles will undergo similar deterioration to other piles during service life of the bridge and will be designed to be accessible for testing during future reuse. This approach may not work for all bridges from an economic point of view, due to the costs of installing additional piling. For relatively small bridges with a limited number of piles, adding even one or two more piles as test piles would not be economically feasible. Conversely, for bridges with large number of piles, installation of a few additional piles or an additional drilled shaft would not increase costs noticeably. The economic case for additional test piles is made by considering the savings from future reuse of the foundation using test piles for detailed assessment. Test piles can be driven near the foundation during initial construction and be available for both mid-life testing and possible substructure widening/strengthening. Driving these piles during initial construction will mitigate concerns about construction related vibrations that could occur if they were driven while the bridge was in service for reuse. Drilled shafts can be used for design verification and acceptance, then buried in-situ to be assessed later.

Pile Groups

Pile groups are commonly found in bridge foundations with driven piles, as more pile can support higher loads, and the inherent redundancy allows for different code factors to be applied. The number of piles and spacing is typically controlled solely by initial costs, however planning for reuse may alter the preferred design. Planning for potential reuse may show that installation of a limited number of additional piles or reconfiguration of piles to a wider or differently shaped footing may allow for reuse without the need for additional piles. Wider spacing of the piles allows for more effective moment resistance (larger moment arm) and higher group efficiency factors that lower the current demands and increase current capacity, respectively.

Installation of Markers into Foundation Elements

Markers, such as passive radio-frequency identification (RFID) tags, can be embedded into concrete elements of bridge piers and foundation elements to store valuable information such as design and construction information (Hamalainen and Ikonen 2008). Since passive tags are not electrically connected, they can have virtually infinite life when embedded into solid concrete. Information is obtained from these tags using a high frequency (HF) or ultra-high frequency (UHF) hand held receiver. Hamalainen and Ikonen (2008) have noted that HF tags outperform ultra-high frequency (UHF) tags when waves are to be transmitted through concrete, and tags far away from the receiver may be difficult to read. Leon-Salas et al. (2011) propose a method of embedding passive RFID tags into concrete elements alongside sensors capable of measuring half-cell potential and linear polarization.

RFID tags can also be used to monitor the stability and performance of riprap used as scour protection by tracking “stones” with embedded sensors (Cassel et al. 2017). Typically, these sensors have been used in short-term applications to validate stream flow modeling, although it is possible this methodology can be adapted to determine the presence and extent of scour. By carefully planning the installation of RFID tags with various functionalities in foundation elements, information related to durability and integrity can be retrieved at regular intervals, like during routine inspection. RFID tags are low-cost and can be installed at different locations in piers and abutments. With the rapid progress of technology in this area, embedded RFID tags can be customized to facilitate inspection, monitor concrete performance, plan maintenance, and inform forensic analysis.

MONITORING

Monitoring of the performance of a foundation can provide useful information that can be used during future reusability assessments. The monitored information can be collected at regular intervals and can be used to continuously analyze changing parameters like settlement, rotation, scour, strains corrosive environment, and more. For example, main river pier settlements were monitored for the Huey P. Long Bridge for a period of 55 years after construction as a part of regular bridge inspection (Modjeski and Masters 2013). This information was extremely useful during the design of foundation strengthening element to support the loading from the widened part of the bridge.

Systems have already been implemented on many bridges to monitor superstructure elements. However, monitoring of foundations is not as common, since there has been more interest in superstructure behavior. A well-planned foundation monitoring program can result in a wealth of information, including load-deformation behavior of different components, settlements, corrosion, overloading, on foundation elements starting from construction to rehabilitation, reuse or rehabilitation in future. After construction, monitoring information can be collected intermittently during routine inspection. Monitoring systems can be customized to collect required information to fulfill many maintenance and safety roles during the service life of the structure. Generally, monitoring techniques can be divided into three generic categories:

- Passive monitoring (for record purposes and for back-analysis when needed)
- Near real-time monitoring (e.g., for construction control)
- Monitoring for safety, when some action or construction could potentially result in an unacceptable result or a dangerous situation developing.

Selecting the proper monitoring instruments and techniques for a new foundation element can decrease the costs and increase the effectiveness of future foundation reuse.

Integrity Monitoring

Thermal Integrity Profiling (TIP)

The Thermal Integrity Profiling (TIP) uses the heat generated during curing of concrete to assess the quality of drilled shafts and of bored, auger cast in place, continuous flight auger or drilled displacement piles (Mullins et al. 2007). TIP evaluates the entire cross-section throughout the length of the foundation by measuring temperature with infrared probes inserted into access tubes, or by thermal wires connected to the rebar cage (Mullins and Kranc 2004). TIP provides results very quickly as the concrete shaft begins to cure. TIP monitoring can identify necking or inclusions as areas of lower concrete temperature and bulging as areas of higher concrete temperature. Variations in cover depth, shape of the shaft, and cage alignment can also be detected. The TIP procedure is covered by ASTM Standard D7949-14 (ASTM 2014). TIP can be performed by installing PVC ducts prior to concrete placement through which thermal probes can be lowered, or through placement of embedded thermal sensors (ASTM 2014). By preserving PVC duct access at the top of the drilled shaft, additional wireline logging probes can be installed later in the life of the drilled shaft. The potential uses of wireline logging probes are discussed in Chapter 4. Embedded thermal sensors can be more difficult to obtain meaningful data from long after construction, as the temperature differential needed to use this method comes from the concrete’s heat of hydration. Still, preserving access to these wires after construction can allow these sensors to be used at a later date.

Monitoring Using Sensors

Although the conventional integrity assessing tools, such as ultrasonic scanning, transient pulse, infrared thermography, and ground penetrating radar, are used regularly for assessment of superstructures and piers, they are not appropriate for in-situ assessment of underground foundation elements. To most effectively increase the potential of future reuse for a newly constructed foundation, there needs to be cyclic monitoring in monthly or yearly intervals. Therefore, using electrical resistance strain gauges, acoustic emission sensors or optical fiber sensors is can be an appropriate tool to this aim.

Immunity to electromagnetic interference, small size and lightweight construction, and ability to measure different response quantities, such as strain, temperature, vibration and specified chemicals, are some of unique advantages of optical fiber sensors compared to other sensors. The optical fiber sensors can also be multiplexed, which means that more than one sensor can be integrated along a single optical fiber. The use of FOSs for detecting strain in concrete structures has been demonstrated by many researchers using a variety of sensing schemes (Kistera et al 2007). Several types of fiber optic strain sensor have been developed, including those based on intensity, polarization, interferometry, and fiber Bragg gratings (FBGs). The FBG sensors are the most popular sensors because of low-cost fabrication techniques and more efficient performance. Underground foundation elements and substructures (e.g., piers) can be instrumented with optical fiber sensors during construction phase. Data from these sensors can be retrieved at intermittent intervals (e.g., during routine inspection) and achieved for use when deciding on major rehabilitation/reuse. Since the primary electronic components are not permanent at the bridge, the sensor setup can operate for very long lifespans.

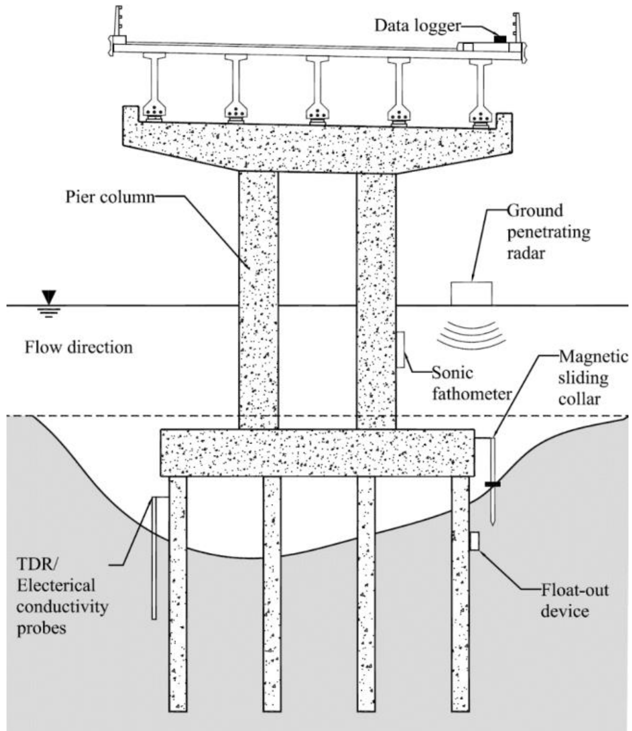
Scour Monitoring

Scour is one the most prominent hazards affecting the integrity of bridge foundations. Monitoring for scour of bridge foundations can be carried out through the following three approaches (Lagasse et al. 2009 and Agrawal et al. 2007):

- Fixed instrumentation
- Portable instrumentation
- Visual monitoring.

Fixed monitors can be placed on a bridge structure, or in the streambed or on the banks near the bridge. More details on the earlier types of fixed scour monitors can be found in the NCHRP Report 396 (Lagasse et al. 1997), which presents state of knowledge and practice for fixed scour monitoring of scour critical bridges, and the corresponding installation, operation, and fabrication manuals (Schall et al. 1997a,b).

A range of instrumentation has been developed to monitor scour hole development. Scour monitoring techniques mostly use underwater instrumentation to measure the progression of scour depths with time. These instruments can be broadly categorized as: single-use devices, pulse or radar devices, buried or driven rod systems, sound-wave devices, fiber-Bragg grating devices and electrical conductivity devices (Prendergast and Gavin 2014). A schematic illustration of typical instrumentation for scour monitoring of a bridge bent is shown in figure 71. A number of researchers have also investigated the effect of scour through the monitoring of the superstructure itself (Prendergast and Gavin 2014). Removal of soil under (or around) the foundation element due to scour will increase stress and consequently reduce stiffness in the remaining soil. Reduction of foundation stiffness yields a change in the frequency of vibration of the structure and therefore, observing changes in vibration frequencies is a potential indirect method for damage identification due to scour.



Original Photo: ©2014 Chinese Academy of Sciences (See Acknowledgments section)

Figure 71. Illustration. Schematic scour monitoring instrumentation

Luh and Liu (2014) have investigated the installation of a scour monitoring with the resolution of 3 ft (1 m) at Yufeng Bridge in Taiwan. Although their system consists of a UHF antenna and transceiver, motion sensor, and HF antenna and transceiver, they have tabulated advantages and limitations of different scour monitoring systems, as shown in table 41 below.

Table 41. Instrumentation for scour monitoring (Luh and Liu 2014)

Instrument	Advantages	Disadvantages or limitations	Cost
RFID	Monitor depth; extensive area monitor, integrated GIS	-	Medium
Bridge mounted sonar	Monitor depth	Mild slope river/estuary; gravel or sandstone interference	Medium
Acoustic Doppler	Monitor velocity and depth	Not applicable to high sediment concentration condition	High
Ground penetrating radar	Monitor depth	More time consuming; specialized training required	High
Fiber Bragg grating	Monitor depth	Avoid stone or rock hitting	High
Numbered bricks	Applicable to high turbulent or rapid flows	Excavation of riverbed required suitable for ephemeral rivers	Low

Durability Monitoring

Corrosion Process Monitoring

As discussed in chapter 5, two of the most important factors to measure when considering reinforcement corrosion is chloride penetration and carbonation (Leung et al. 2008). Typically, the chloride concentration is measured directly from concrete samples taken from various depths (Song et al. 2008). However, coring can be structurally unacceptable, expensive to perform, and only provide limited data. Therefore, nondestructive methods, where corrosion monitoring sensors are embedded in concrete of the foundation may be preferable over future sampling and testing (Agrawal et al. 2009). Generally, the existing methods of corrosion monitoring can be classified into six main categories as follows (Zaki et al. 2015):

- Visual inspection
- Electrochemical methods (*i.e.*, open circuit potential monitoring, resistivity method, polarization resistance, galvanostatic pulse method, electrochemical noise)
- Elastic wave methods (*i.e.*, ultrasonic pulse velocity, acoustic emission, and impact echo)
- Electromagnetic methods (*i.e.*, ground penetrating radar)
- Optical sensing methods (*i.e.*, fiber Bragg grating FBG)
- Infrared thermography

To measure chloride concentration remotely, Lesthaeghe (2013) has utilized radio-frequency identification (RFID) based technology to passively report the level of chloride ingress in bridge decks. SRI International has developed a wireless sensor for monitoring the level of chloride ingress into concrete bridge decks. The technology combines a chloride sensor and an RFID chip that can be accessed wirelessly to provide chloride concentrations at the sensor location (Watters et al. 2003). Virginia Technologies has developed embeddable corrosion-monitoring instrument (ECI) which is capable of measuring parameters important to long-term corrosion monitoring, including linear polarization resistance (LPR), open circuit potential, resistivity, chloride ion concentration, and temperature. In addition to embeddable sensors mentioned, several other sensors are in development could be implemented for field monitoring of bridges. Agrawal et al. (2009) have summarized the sensing technologies for remotely monitoring of long-term corrosion process in the bridges.

Capacity Monitoring

The bearing capacity of deep foundations is function of foundation material, geometric properties of foundation (length, embedment depth and cross-sectional area), soil layers type and their strength properties, driving process (for driven piles), ground water table, and the type of applied loads. In general, among the factors affecting foundation bearing capacity, after installation/construction of driven piles or drilled shafts, only foundation integrity and soil mechanical properties can change. To take the changes of pile capacity over time into account, long-term resistance, available during the entire foundation design life (R_n), is typically used in design of a new foundation. Long-term resistance is defined as the minimum pile bearing capacity that would always be available to support the applied pile factored axial loads during the entire design life of the bridge (Abu-Hejleh et al. 2013). Short-term resistance (R_{nre}) is estimated from the side and base resistances of all the soil layers around the pile, including contributions from those layers that could eventually contribute to geotechnical resistance losses due to downdrag or scour. Resistance at end of driving (EOD) is estimated from R_{nre} and the given time-dependent changes in resistance after driving (e.g. setup).

Since the pile capacity may increase after the end of driving, considering the potential for soil setup can allow for more accurate determination of capacity. Figure 72 shows the increase in pile capacity with time, with t0 being the time at which equilibrium conditions have been re-established.

Although pile load test specifications often require a minimum waiting period of seven days between installation of the last pile and the test (NYSDOT 2015), some researchers show that the pile capacity increase in soil may continue up to a time longer that 200 days (Alawneh et al. 2009). The long-term ultimate pile capacity may range between 50 and 1000 percent of the end-of- driving capacity (Rausche et al. 2004, Samson and Authier 2011). This is illustrated in figure 72 through the capacity increase in Phases 2 and 3.

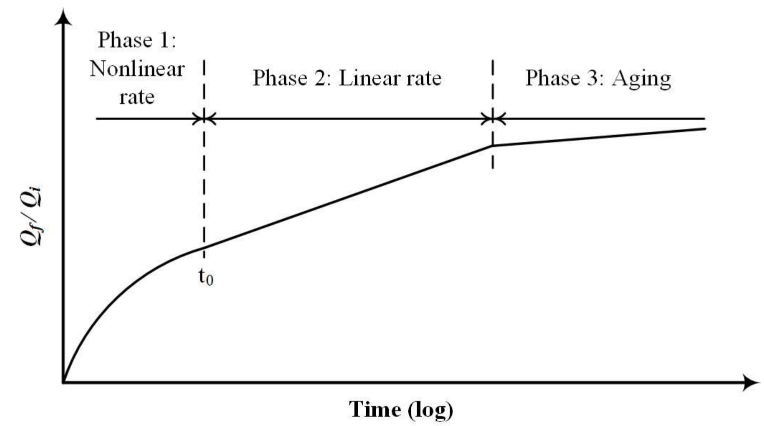


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Figure 72. Graph. Idealized capacity change of driven pile with time

Therefore, it is possible to gain increased capacity from pile foundation at time of reuse. This increased capacity can be accounted for through monitoring of the side friction, tip resistance and strain in piles during the service life of the foundation. This measurement can be done intermittently during biennial inspection and data can be collected intermittently and can be stored for analysis during foundation reuse investigation. Driven piles and drilled shafts can be instrumented using a number of techniques to study their behavior during the load test or operation of the bridge (Glisic et al. 2002, Tsumoto et al. 2004, Kister et al. 2007, Civjan et al. 2013, Li et al. 2014). Piles can be instrumented to measure strain in a pile using strain gages or load cells, pore water pressure using a piezometer, vertical deformation in the soil using extensometer or settlement gages, horizontal deformation using inclinometers and pile–soil friction distributions using distributed strain measurements. Table 42 summerizes the most common sensors used for the instrumentation of a pile or drilled shaft during the construction period so that the necessary information can be collected on performance monitoring and potential for future reuse.

Figure 73 shows a shematic diagram of an instrumented pile/shaft. This instrumentation plan can be used both for static load test as well as long term monitoring.

DOCUMENTATION

Whenever the reuse of an existing foundation is considered, the information and data on the existing foundation play a vital role in the investigation and planning of reuse. Availability and easy access to the design information related to design (preliminary and final), construction, monitoring, maintenance, and repairs can decrease the time and costs of reuse investigation. These potential savings make preservation of the data from the planning, design, construction and control phases (table 43) highly important.

Regularly, in a bridge design project, enough geotechnical investigations (i.e. laboratory and in-situ testing) for estimation of soil mechanical and strength properties are performed to be used in design of substructure/foundation of the bridge. However, the results of investigation and related documents are not recorded and saved either in the uniform standard format or for a long time to be implemented after a while in reusability assessment of a foundation. Geo-Institute of American Society of Civil Engineers (ASCE) has development an XML-based geospatial standard schema for the transfer of geotechnical and geoenvironmental data within an organization or between multiple organizations. DIGGS also provides a standardized format for summarizing geotechnical information and sharing this information between individuals and organizations.

Table 42. Various instruements for monitoring of a pile

<table>
<tr><th>Parameter</th><th>Instrument</th><th>Sensor Type</th><th>Usage</th></tr>
<tr><td rowspan="3">Position/ Deformation</td><td>Geodetic</td><td>Optical laser</td><td>Foundation surface</td></tr>
<tr><td rowspan="2">Extensometer</td><td>Vibrating wire</td><td>Pile</td></tr>
<tr><td>Electrical</td><td>Soil layer</td></tr>
<tr><td rowspan="2">Inclination</td><td>In-Place Inclinometer</td><td>Vibrating wire</td><td>Pile</td></tr>
<tr><td>Inclinometer, Smart rod</td><td>Fiber Optic</td><td>Pile</td></tr>
<tr><td rowspan="3">Distance</td><td rowspan="2">Strain gauge</td><td>Vibrating wire</td><td>Rebar, Pile</td></tr>
<tr><td>Fiber Optic</td><td>Pile</td></tr>
<tr><td>Arc Weldable</td><td>Vibrating wire</td><td>Steel piles</td></tr>
<tr><td rowspan="2">Pressure</td><td rowspan="2">Load Cell</td><td>Electric</td><td>Pile head</td></tr>
<tr><td>Hydraulic and electrical</td><td>Pile head and tip, soil interface</td></tr>
<tr><td rowspan="3">Pore-water Pressure</td><td rowspan="3">Piezometer</td><td>Vibrating wire</td><td rowspan="3">Monitor excess pore-water pressures</td></tr>
<tr><td>Pneumatic</td></tr>
<tr><td>Standpipe</td></tr>
<tr><td rowspan="3">Settlement</td><td>Settlement Cells</td><td>Vibrating wire</td><td>Pile</td></tr>
<tr><td>Settlement extensometer</td><td>Vibrating wire</td><td>Pile</td></tr>
<tr><td>Settlement points</td><td>Vibrating wire</td><td>Pile</td></tr>
</table>

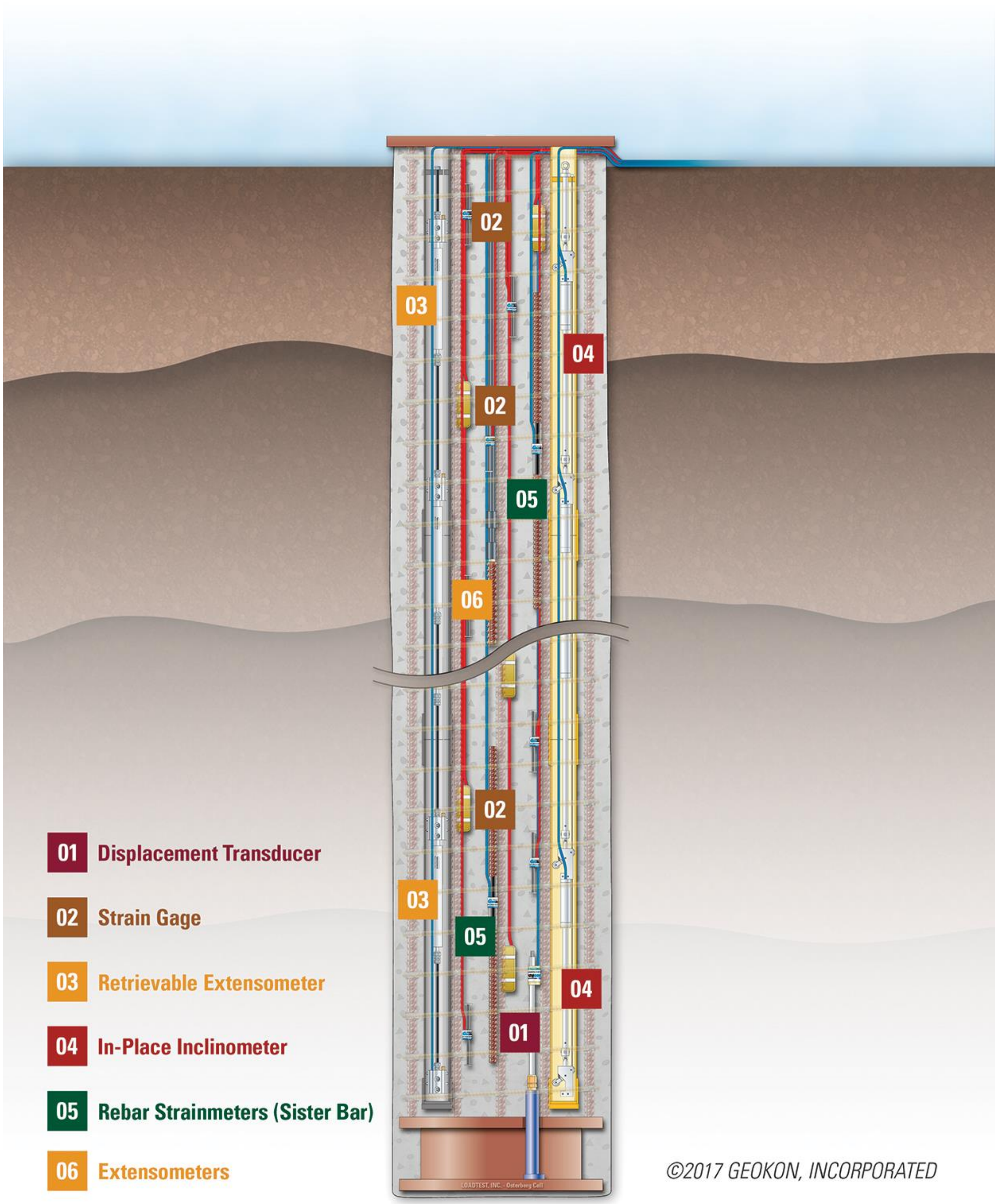

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Figure 73. Illustration. Schematic pile testing instrumentation

Table 43. Recommended information preservation for future foundation reuse

Planning Phase

- Geological data
- Geotechnical information
- Groundwater data
- Hydrolic and scour reports
- Feasibility study report
- Enviromental impact assessment report

Design Phase

- Geotechnical report
- Borehore logs (e.g. soil classification, visual description, ground water level, etc.)
- Lab test data (e.g. consolidation, gradation, strength, etc.)
- In-situ test data (e.g. SPT, CPT, vane shear, pressuremeter, etc.)
- Geophysical test (e.g. shear wave velocity, etc.)
- Design codes
- Design calculation sheets
- Bearing capacity
- Settlement limitation
- QA/QC reports

Construction Phase

- Construction drawings
- Concrete testing data
- Pile driving program
- Plant and equipment
- Pile driving data
- Drilled shaft data
- Monitoring data
- Pile integrity testing results
- Pile load testing data

Control Phase

- As-built drawing
- Inspection and load rating reports
- Load permit records
- Maintenance records
- Scour monitoring reports
- Scour maintenace reports
- Quality assessment and quality control

ACKNOWLEDGMENTS

Support of the following people who provided information related to foundation reuse, case studies, or helped with review of sections of the report, is sincerely acknowledged.

- Darrin Beckett — Kentucky Transportation Cabinet
- Andrew Boeckmann — University of Missouri
- Leslie Connelly — American Society of Civil Engineers
- Peter J Connors — MassDOT
- Jerry DiMaggio — Applied Research Associates
- Tim Dovan — American National Standards Institute
- Scott J. Fisher — Virginia DOT
- Zhengzheng "Jenny" Fu — Louisiana DOT
- Hota GangaRao — West Virginia University
- Brian Hanks — North Carolina DOT
- Dennis Heckman — Missouri DOT
- Kevin Hetrick — Indiana DOT
- Randy Hitt — Missouri DOT
- Houston Walke — Tennessee DOT
- Jeff C. Jones — Tennessee DOT
- Nat Kasbekar — New Jersey DOT
- Laura Krusinski — MaineDOT
- Sharon Lamberton — The National Academies of Sciences
- Mahmoud Lardjane — Canadian Society for Civil Engineering
- Songqing Lin — Institute of Rock and Soil Mechanics
- Maureen Mandel — Deep Foundations Institute
- Angela Matthews — American Concrete Institute
- John Moran — MassDOT
- Mohammed Mulla — NCDOT
- Mete Oner — EJGE
- Pankesh Patel — New Jersey DOT
- Wayne J. Seger — Tennessee DOT
- Trever Wang — Colorado DOT
- Andrew M. Zickler — Virginia DOT

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REFRENCES

AASHTO (2010). R 27-01 - *Assessment of Corrosion of Steel Piling for Non-Marine Applications.* American Association of State Highway and Transportation Officials (AASHTO).

AASHTO (2011). *Guide Specifications for LRFD Seismic Bridge Design*. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2014). *LRFD Bridge design specifications*. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2016). *Manual for Bridge Evaluation*. American Association of State Highway and Transportation Officials, Washington, D.C.

Abu-Hejleh, N., Kramer, W.M., Mohamed, K., Long, J.H., and Zaheer, M.A. (2013). *Implementation of AASHTO LRFD Design Specifications for Driven Piles*, FHWA-RC-13-001. Federal Highway Administration Resource Center, Matteson, IL.

ACI. (1992). ACI 209R-92: *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures.* Report of ACI Committee 209, American Concrete Institute, Farmington Hills, MI.

ACI. (1997) ACI 546.1R *Guide for repair of concrete bridge superstructures.* Report of ACI Committee 546, American Concrete Institute, Farmington Hills, MI.

ACI. (2001). ACI 222R-01: *Protection of Metals in Concrete against Corrosion* (Reapproved 2010). Report of ACI Committee 222, American Concrete Institute, Farmington Hills, MI.

ACI. (2003). ACI 437R-03: *Strength Evaluation of Existing Concrete Buildings*, Report of ACI437, American Concrete Institute, Farmington Hills, MI.

ACI. (2016). ACI Committee 201: *Guide to Durable Concrete,* American Concrete Institute, Farmington Hills, MI.

Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W., Hartmann, J. (2011). *Geosynthetic Reinforced Soil Integrated Bridge System,* FHWA-HRT-11-027, Federal Highway Administration, Washington, D.C

Agrawal, A. K., Khan, M.A., Yi, Z. (2007). *Handbook of Scour Countermeasures Design,* FHWA-NJ-2005-027, New Jersey Department of Transportation, Trenton, NJ

Agrawal, A. K., Yi, Z., Alampalli, S., Ettouney, M., King, L. Hui, K., and Patel, M. (2009). "Remote Corrosion Monitoring Systems for Highway Bridges." 14(4), pp.152-158, ASCE. Pract. Period. Struct. Des. Constr. Reston, VA.

Agrawal, A.K. and Gong, X. (2015), "Numerical Simulation of Fire Damage to a Long-Span Truss Bridge", J. Bridge Eng., Vol. 20(10): 04014109.

Agrawal, A.K. and Gong, X. (2016), "Safety of Cable-Supported Bridges during Fire Hazards", J. Bridge Eng., Vol. 21(4): 04015082.

Agrawal, A.K., El-Tawil, S., Cao, R., Xu, X., Chen, X. and Wong, W. (2018). A Performance Based Approach for Loading Definition of Heavy Vehicle Impact Events. Federal Highway Administration, Office of Bridge Technology, Washington, DC. 20590.

Aktan, H. and Attanayake U. (2015). *Research on Evaluation and Standardization of Accelerated Bridge Construction Techniques*. Report: RC-1618A. Michigan Department of Transportation, Research Administration, Lansing, MI.

Alawneh, A.S., Nusier O.K., and Awamleh, M.S. (2009). "Time Dependent Capacity Increase for Driven Pile in Cohesionless Soil." Volume 3, No. 1, Jordan Journal of Civil Engineering. Amman, Jordan.

Andrawes, B. and Caiza, P. (2012). "Bridge Timber Piles Load Rating Under Eccentric Loading Conditions." Vol. 17, No. 4, pp. 700-710. ASCE Journal of Bridge Engineering. Reston, VA.

Armour, T., Groneck, P., Keeley, J., and Sharma, S. (2000). *Micropile Design and Construction Guidelines Implementation Manual Priority Technologies Program (PTP) Project,* Report No. FHWA-SA-97-070, Federal Highway Administration (FHWA), Washington, D.C.

Arneson, L.A., Zevenbergen, L.W., Lagasse, P.F., Clopper, P.E. (2012). HEC-18 *Evaluating Scour at Bridges, Fifth Edition.* FHWA-HIF-12-003. Federal Highway Administration, Office of Bridge Technology, Washington, DC. 20590

Arurkar, T. P. (2005). *Accelerated Construction Decision Making Process*. Master of Science Thesis, University of Cincinnati, OH.

Asher B., Beckett D., Zang S.D. (2010). *Milton-Madison Bridge Geotechnical Overview*. Milton-Madison US 421 Bridge Project, Kentucky Transportation Cabinet and Michael Baker Corporation.

ASTM (2012a). *ASTM C295/C295M-12* "Standard Guide for Petrographic Examination of Aggregates for Concrete", ASTM International, West Conshohocken, PA.

ASTM (2012b). *ASTM C823/C823M-12* "Standard Practice for Examination and Sampling of Hardened Concrete in Constructions.", ASTM International, West Conshohocken, PA.

ASTM (2012c). *ASTM D2899-12* "Standard Practice for Establishing Allowable Stresses for Round Timber Piles.", ASTM International, West Conshohocken, PA.

ASTM (2012d). *ASTM C1532/C1532M-12 "*Standard Practice for Selection, Removal, and Shipment of Manufactured Masonry Units and Masonry Specimens from Existing Construction.", ASTM International, West Conshohocken, PA.

ASTM (2012e). *ASTM D4580/D4580M-12* "Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding." ASTM International, West Conshohocken, PA.

ASTM (2012f). *ASTM C1202-12* "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." ASTM International, West Conshohocken, PA.

ASTM (2012g). *ASTM C1152/C1152M-04* “Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete.” ASTM International, West Conshohocken, PA.

ASTM (2013a). *ASTM C805/C805M-13a* “Standard Test Method for Rebound Number of Hardened Concrete”, ASTM International, West Conshohocken, PA.

ASTM (2014a). *ASTM C856-14* “Standard Practice for Petrographic Examination of Hardened Concrete”, ASTM International, West Conshohocken, PA.

ASTM (2014b). *ASTM C469/C469M-14* “Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression.”, ASTM International, West Conshohocken, PA.

ASTM (2014c) *ASTM D7949-14* “Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations”, ASTM International, West Conshohocken, PA.

ASTM (2015a). *ASTM D6635-15* “Standard Test Method for Performing the Flat Plate Dilatometer”, ASTM International, West Conshohocken, PA.

ASTM (2015b). *ASTM C1587/C1587M-15 “*Standard Practice for Preparation of Field Removed Manufactured Masonry Units and Masonry Specimens for Testing.”, ASTM International, West Conshohocken, PA.

ASTM (2015c). *ASTM C876-15* “Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete.”, ASTM International, West Conshohocken, PA.

ASTM (2015d). *ASTM C1218/C1218M-15* “Standard Test Method for Water-Soluble Chloride in Mortar and Concrete.”, ASTM International, West Conshohocken, PA.

ASTM (2016a). *ASTM D5882-16* “Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations”, ASTM International, West Conshohocken, PA,

ASTM (2016b). *ASTM D6760-16* “Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing”, ASTM International, West Conshohocken, PA

ASTM (2016c). *ASTM C42/C42M-16* “Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.”, ASTM International, West Conshohocken, PA.

ASTM (2016d). *ASTM D2555-16* “Standard Practice for Establishing Clear Wood Strength Values”, ASTM International, West Conshohocken, PA

ASTM (2016e). *ASTM D6760-16* “Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing,” ASTM International, West Conshohocken, PA

ASTM (2017). *ASTM D6817/D6817M-17* “Standard Specification for Rigid Cellular Polystyrene Geofoam.” West Conshohocken, PA: ASTM International.

ATC/MCEER. (2002). *Comprehensive Specification for the Seismic Design of Bridges* NCHRP Report 472, Transportation Research Board of the National Academies, Washington, D.C.

Axtell, P.J. and Siegel, T.C., Brown, D. (2014). “Sustainability and Reuse of Foundations for Hurricane Deck Bridge.” Proceedings of the 39th Annual Conference on Deep Foundations, Deep Foundation Institute, Atlanta, GA.

Bazant, Z. P. (1979) “Physical Model for Steel Corrosion in Concrete Sea Structures—Theory and Application,” *Journal of the Structural Division*, 105, 2, # ST6.

Bell, A., K. Soga, Y. Ouyang, J. Yan, and F. Wang. (2013). “The Role of Fibre Optic Instrumentation in the Re-Use of Deep Foundations.” pp. 1863–1866, Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, France.

Bentur A., Diamond S., Berke N.S. (1997). *Steel Corrosion in Concrete*. E & FN SPON, London.

Berg, R.R., Christopher, B.R., and Samtani, N.C. (2009). *Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,* FHWA-IF-99-015, Federal Highway Administration, Washington, D.C.

Bertolini, L., Elsener, B., Pedeferri, P., Redaelli, E., and Polder, R. B. (2013). *Corrosion of Steel in Concrete: Prevention, Diagnosis, Repair.* 2nd Edition. Wiley-VCH Verlag GmbH & Co. KGaA. Weinhem, Germany.

Bica, A. V. D., Salgado, R., Kim, D., Seo, H., and Prezzi, M. (2014). “Instrumentation and axial load testing of displacement piles,” vol. 167, pp. 238–252, Proceedings of the Institution of Civil Engineers - Geotechnical Engineering

Boeckmann, A.Z. and Loehr, J.E. (2017) *Current Practices and Guidelines for the Reuse of Bridge Foundations*. NCHRP Synthesis 505, Transportation Research Board of the National Academies, Washington, D.C.

Bradshaw, A.S., and Baxter, C.D.P. (2006). *Design and Construction of Driven Pile Foundations – Lessons learned on the Central Artery/Tunnel Project*, FHWA-HRT-05-159, Office of Infrastructure Research and Development, Federal Highway Administration, Washington, D.C

Briaud J.L., Chen H., Li Y., Nurtjahyo P., Wang J. (2005). SRICOS-EFA method for contraction scour in fine-grained soils. Journal of Geotechnical and Geoenvironmental Engineering 131(10)

Briaud JL, Ting F, Chen HC, Cao Y, Han SW, Kwak K. (2001). “Erosion function apparatus for scour rate predictions.” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol 127, Issue 2, pp. 105-117. Reston, VA.

Bridge Grid Flooring Manufacturers Association (BGFMA) (2017) Grid Reinforced Concrete and Exodermic® Bridge Decks. Bridge Grid Flooring Manufacturers Association, North Baltimore OH 45872 http://www.bgfma.org/resources/pdf/Brochure/BridgeDecksBro_WebVrs.pdf

Brown, D.A., Dapp, S.D., Thompson, W.R., Lazarte, C.A. (2007). Geotechnical Engineering Circular (GEC) No. 8 *Design and Construction of Continuous Flight Auger (CFA) Piles,* Report No. FHWA-HIF-07-03, Federal Highway Administration (FHWA), Washington, D.C. 20590

Brown, D.A., Turner, J.P., and Catelli, R.J. (2010). Geotechnical Engineering Circular (GEC) No. 10 *Drilled Shafts: Construction Procedures and LRFD Design Methods,* NHI Course No. 132014, Report No. FHWA NHI-10-016, Federal Highway Administration (FHWA), Washington, D.C. 20590.

Brown, D. A. (2014). *Issues of Risk and Liabilities Relating to Reuse of Existing Foundations*, presentation to the 93rd Annual Meeting of the Transportation Research Board, Washington, D.C.

Browne, T. M., Collins, T.J., Garlich, M.J., O'Leary, J.E., Stromberg, D.G., Heringhaus, K.C. (2010). *Underwater Bridge Inspection,* FHWA-NHI-10-027. Office of Bridge Technology, Federal Highway Administration, Washington, D.C.

Bruce, D. A. and Juran, I. (1997). *Drilled and Grouted Micropiles, State-of-Practice Review*, Report No. FHWA-RD-96-016/019, Federal Highway Administration (FHWA), Washington, D.C.

Buckle, I., Friedland, I., Mander, J., Martin, G., Nutt, R., Power, M. (2006) *Seismic Retrofitting Manual for Highway Structures: Part 1 — Bridges,* FHWA-HRT-06-032, Office of Infrastructure Research and Development, Federal Highway Administration (FHWA), McLean, VA 22101

Bungey, John H., Michael G. Grantham, and Stephen Millard. (2006). *Testing of Concrete in Structures: Fourth Edition* CRC Press.

Butcher, A.P., Powell, J.J.M., and Skinner, H.D. (2006). *Reuse of Foundations for Urban Sites: A Best Practice Handbook*, IHS BRE Press, Hertfordshire, United Kingdom.

CalTrans (2015). *Corrosion Guidelines Version 2.1*, California Department of Transportation Division of Engineering Services, Sacramento, CA.

Cassel, M., Dépret, T. and Piégay, H. (2017), "Assessment of a new solution for tracking pebbles in rivers based on active RFID", Earth Surface Processes and Landforms.

Central Federal Lands Highway Division (CFLHD). (2015) "Structure Selection Report, Lake Mary Road." Willow Valley Creek Bridge AZ FLAP 234(3). Federal Highway Administration. Washington, DC.

Chapman, T., S. Anderson, and J. Windle. (2007). Reuse of Foundations, Report C653, CIRIA, London, United Kingdom.

City of Calgary. (2015). *Crowchild Trail Study.* The City of Calgary, CA. last accessed on March 15, 2018 at:

Civjan, S.A., Kalayci, E., Quinn, B. H., Breña, S. F. and Allen, C. A. (2013). "Observed integral abutment bridge substructure response," Eng. Struct., vol. 56, pp. 1177–1191.

Collin, J. G. and Jalinoos, F. (2014). "*Foundation Characterization Program: TechBrief #1— Workshop Report on the Reuse of Bridge Foundations.*" Publication FHWA-HRT-14-072, Federal Highway Administration, Washington, D.C.

Collin, J.G. (2015) "Timber Pile Design and Construction Manual", Timber Pile Council, Southern Pressure Treater's Association, Starkville, MS. 39760

Collin, J.G., Mullins, G., Lewis, C., and Winters, D. (2014). State of the Practice and Art for Structural Health Monitoring of Bridge Substructures. FHWA-HRT-09-040. Federal Highway Administration. McLean, VA

Constructability Review Guidelines (2011). Idaho Transportation Department

Cooper, D., Grey, S., Raymond, G., and Walker, P. (2005). Project Risk Management Guidelines: Managing Risk in Large Projects and Complex Procurements. Chichester: John Wiley & Sons, Ltd.

Dahlberg, J., Phares, B., and Klaiber, W. (2015) *Development of Cost-Effective Timber Bridge Repair Techniques for Minnesota* MN/RC 2015-45A, MNDOT, St. Paul, MN 55155

Das, S. (2010) *"Design Considerations in the Rehabilitation of a Historic Arch Bridge across the Tennessee River in Knoxville, Tennessee"* ACI Spring 2010 Convention, 2010, Chicago, IL

Davis, C., Sammarco, E., Williamson, E. (2017) *Bridge Design Security Manual* FHWA-HIF-17-032. Federal Highway Administration, Washington, DC, 20590.

Davis, N. T., Hoomaan, E., Sanayei, M., Agrawal, A. K., and Jalinoos, F. (2018[a]). "Integrated Superstructure-Substructure Load Rating for Bridges with Foundation Movements." Journal of Bridge Engineering, 23(5), 1–12.

Davis, N.T., Sanayei, M., Agrawal, A.K. Jalinoos, F. (2018[b]) "*Determining the Capacity of Reused Bridge Foundations from Limited Information"*, ASCE Journal of Bridge Engineering, in press

Davisson, M.T. (1972). High Capacity Piles, Proceedings, Soil Mechanics Lecture Series on Innovations in Foundation Construction. American Society of Civil Engineers, ASCE, Illinois Section, Chicago, 81-112.

Dhillon, B.S. (1989). Life Cycle Costing: Techniques, Models and Applications. Gordon and Breach Science Publishers, New York

Dow, T. and Sokoloff. S. (2013). *Bay of Quinte Skyway (Site No. 11-245) VE Study. CSVA Conference*

Dugarte, M., Sagüés, A.A. (2009). "Galvanic Point Anodes for Extending the Service Life of Patched Areas Upon Reinforced Concrete Bridge Members

Emerson, R. N. (2004). "In Situ Repair Technique for Decayed Timber Piles." *Structures 2004: Building on the Past, Securing the Future*. Restoration and Repair of Existing Structures Section. Vol. 137, No. 65.

FHWA. (2002). "Life-Cycle Cost Analysis Primer" *FHWA IF-02-047*, US Department of Transportation, Office of Asset Management

FHWA. (2017), *National Bridge Inventory*, Deficient Bridges by Highway System in 2016. Federal Highway Administration. Last accessed November 10, 2017.

FHWA. (2017). “Infrastructure Voluntary Evaluation Sustainability Tool (INVEST),” Federal Highway Administration, Washington, DC. Last accessed October 17, 2017.

FHWA. (2017). “NDE web manual,” Federal Highway Administration, Washington, DC. Last accessed October 17, 2017.

Florida International University (FIU). (2017) *ABC Project Database*. Accelerated Bridge Construction University Transportation Center. Florida International University. Last accessed on 3/15/2018.

Gajewska, E., and Ropel, M. (2011) “Risk Management Practices in a Construction Project — a case study” Master’s Thesis, Chalmers University of Technology, Göteborg, Sweden

Geosciences Testing and Research, Inc. (GTR) (2014). “Jackson Road over Route 2 Evaluation of Existing Timber Pile Capacity Report. Letter Report to Lin Associates Incorporated.

Ghosn, M., Moses, F., Wang, J. (2003). NCHRP Report 489 *Design of Highway Bridges for Extreme Events.* Transportation Research Board. Washington, DC, 20001

Gingery, J.R., Franke, K.F., Meneses J.F., and Zafir, Z. (2015). North Torrey Pines Bridge Seismic Retrofit: Part I, Seismic Hazard and Geotechnical Design. Earthquake Spectra. Vol 31, Issue 4.

Glisic, B., Inaudi, D., and Nan, C. (2002). “Pile Monitoring with Fiber Optic Sensors During Axial Compression, Pullout, and Flexure Tests,” Transp. Res. Rec., vol. 1808, no. 02, pp. 11–20,

Google®. (2018). *Google® Map™*, Street View, CA, last accessed June 1, 2018.

Guades, E., Aravinthan, T., Islam, M. and Manalo, A. (2012), “A review on the driving performance of FRP composite piles”, Composite Structures Volume 94, Issue 6, May 2012, Pages 1932–1942.

GZA GeoEnvironmental, Inc. (2014). “Pile Condition Assessment and Pile Load Test Results, Route 2A Bridge over Mattawamkeag River, Haynesville, Maine,” Letter Report to the Maine DOT, rev. May 16, 2014.

Hamalainen, H. and Ikonen, J. (2008) Requirements for RFID tagging process of concrete elements in building project. 16th International Conference on Software, Telecommunications and Computer Networks, SoftCOM 2008. Split, Croatia.

Han, J. et al. (2003). Design of fiber-reinforced polymer composite piles under vertical and lateral loads. Transportation Research Record: Journal of the Transportation Research Board, 1849(1), 71-80.

Hannigan, P.J., Goble, G.G., Likens, G.E., and Rausche, F. (2006). “Design and Construction of Driven Pile Foundations.” FHWA-NHI-05-042, National Highway Institute, Federal Highway Administration, Washington, D.C

Hawk, H. (2003). NCHRP Report 483. *Bridge Life-Cycle Cost Analysis*. Transportation Research Board of National Academies. National Engineering Technology Corporation Arlington Heights, IL.

Hischier R., Weidema B., Althaus H.-J., Bauer C., Doka G., Dones R., Frischknecht R., Hellweg S., Humbert S., Jungbluth N., Köllner T., Loerincik Y., Margni M. and Nemecek T. (2010) Implementation of Life Cycle Impact Assessment Methods. ecoinvent report No. 3, v2.2. Swiss Centre for Life Cycle Inventories, Dübendorf.

Hong, T., Ji, C.Y., Jang, M., Park, H. (2014). "Assessment Model for Energy Consumption and Greenhouse Gas Emissions during Building Construction." J. Manage. Eng. 30(2): 226-235.

Hopwood, T. and D. W. Prine. (1987). *Acoustic Emission Monitoring of in-Service Bridges UKTRP-87-22*. Frankfort, KY 40622: Kentucky Transportation Cabinet.

Hunt, B.E. (2009). NCHRP Synthesis 396c, Monitoring Scour Critical Bridges, A Synthesis of Highway Practice,

Husdal, J. (2009). Supply Chain Risk Management in six steps. Last accessed November 10, 2017.

Illinois DOT (IDOT). (2011) Bridge Condition Report Procedures & Practices. Bureau of Bridges and Structures Division of Highways. Springfield, MA.

Imhof, D. (2004). *Risk assessment of existing bridge structures.* Ph.D. Dissertation. University of Cambridge. Cambridge, UK.

Islam, M., Daily, S.F. (2006) "Use of Electrochemical Techniques to Assess Performance of Corrosion Protection Systems for Reinforce Concrete Structures – A Review" *Journal of ASTM International, Vol. 3, No. 2*, ASTM International

ISO 14040:2006 (2006) Life cycle assessment - Principle and Framework

Jacobs Engineering Group. (2013). Initial Scoping Report of Lake Mary Road Widening and Bridge 3R. Central Federal Lands Highway Division. Federal Highway Administration. Washington, DC.

Jalinoos, F., Olson, L. D., and Aouad, M. F. (1996). Determination of unknown depth of bridge foundations using two nondestructive seismic methods. Procedures, Symposium on the Application of Geophysics to Engineering and Environmental Problems (SAGEEP), Environment and Engineering Geophysical Society, Tulsa, OK.

Jalinoos, F, Mekic, N., Grimm, R.E., Hanna, K. (2005). Defects in Drilled Shaft Foundations – Identification, Imaging, and Characterization. Federal Highway Administration, Central Federal Lands Highway Div., Publication No. FHWA-CFL/TD-05-002.

Jalinoos, F. (2015). Reusing Bridge Foundations. Public Roads, Vol. 79, No. 3. Last accessed November 10, 2017.

Jalinoos, F., Mulla, M., Schaefer, V.R. (2016). Foundation Reuse and Enhancement: A Viable Option for Bridge Widening and Replacement Projects. Geo-Strata—Geo Institute of ASCE, Vol. 20, Issue 3, Pg. 52-57.

Jalinoos, F, Tran, K. T., Nguyen, T. D., and Agrawal, A. K., (2017). Evaluation of Bridge Abutments and Bounded Wall Type Structures with Ultraseismic Waveform Tomography. ASCE Journal of Bridge Engineering, Volume 22 Issue 12.

Johnson, N. and Creveling, M. (2015). "Engineering Elegant Solution." Civil Engineering Magazine. The American Society of Civil Engineers. pp. 59-65.

Johnson, N.S., Creveling, M.L. and Gazaway K.J. (2015). North Torrey Pines Bridge Seismic Retrofit: Part II, Structural Analysis and Design. Vol 31, Issue 4.

Juran, J. and Komornik, K. Behavior of Fiber-Reinforced Polymer Composite Piles Under Vertical Loads. FHWA-HRT-04-107. The Federal Highway Administration. McLean, VA. 2006

Kistera,G., Wintera, D., Gebremichaelb, Y.M., Leightonb,J., Badcocka, R.A., Testerc, P.D., Krishnamurthya,S., Boyleb, W.J.O., Grattanb, K.T.V., Fernando, G.F. (2007). Methodology and integrity monitoring of Foundation Concrete Piles Using Bragg Grating Optical Fibre Sensors. Engineering Structures, Vol. 29(9):2048–2055.

Kleinfelder/Simon Wong Engineering (2015). "Revised Final Seismic Strategy Report of Georgia Street Bridge at University Avenue." Prepared for The City of San Diego. CA.

Komurka, V.E., and Wagner, A.B., and Edil, T.B. (2003). Estimating Soil/pile Set-up. Final Report #0092-00-14:, submitted to the Wisconsin Department of Transportation.

Laefer, D.F. and Farrell K.A. (2015). A hybrid method for foundation reuse evaluation. Environmental Geotechnics. Institution of Civil Engineering,

Lagasse, P.F., C.F. Nordin, J.D. Schall, and G.V. Sabol. (1991). "Scour Monitoring Devices for Bridges," Transportation Research Record 1290, Transportation Research Board, National Research Council, Washington, D.C., pp. 281–294.

Lagasse, P.F., Clopper, P.E., Pagán-Ortiz, J.E., Zevenbergen, L.W., Arneson, L.A., Schall, J.D., and Girard, L.G. (2009). HEC-23 *Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance*. *Volumes 1 and 2, Third Edition*. FHWA-NHI-09-111/112. Federal Highway Administration, Office of Bridge Technology, Washington, DC. 20590

Lagasse, P.F., Clopper, P.E., Zevenbergen, L.W., Spitz, W.J., Girard, L.G. (2010) *Effects of Debris on Bridge Pier Scour* NCHRP Report 653, Transportation Research Board, Washington DC, 20001

Lamson Engineering. (2011) Geotechnical Report for Cedar Street over Route 9 Bridge Superstructure Replacement Project, Massachusetts Department of Transportation.

Lazarte, C.A., Robinson, H., Gómez, J.E., Baxter, A., Cadden, A., Berg, R. (2015) Geotechnical Engineering Circular (GEC) No. 7 *Soil Nail Walls,* Report No. FHWA-IF-14-007, Federal Highway Administration (FHWA), Washington, D.C. 20590

LeGrand, L. (2015) *Keeping Virginia Moving* FHWA-HRT-15-005, Public Roads, Vol. 79, No. 1, Federal Highway Administration

Lehtonen, J. and Ville-Veikko, H. (2010). *Classification of Micropile Underpinning Methods Exemplified by Projects in Turku*, Electronic Journal of Geotechnical Engineering, 295 - 309

Leon-Salas, W., Kanneganti, S., Halmen, C. (2011) Development of a smart RFID-based corrosion sensor. *2011 IEEE SENSORS Proceedings*, Limerick, 2011, pp. 534-537. doi: 10.1109/ICSENS.2011.6127274

Lesthaeghe,T., Frishman, T., and Holland, S.D. RFID. (2013). Tags for Detecting Concrete Degradation in Bridge Decks. InTrans Project 11-404. Iowa Department of Transportation. Ames, IA.

Leung, C.K.Y., Wan, K.T. and Chen, L. (2008). A Novel Optical Fiber Sensor for Steel Corrosion in Concrete Structures. Sensors Journal. 8(3): 1960–1976.

Li, E., Morozevych, S., and Lo, C. (2014). "Underpinning of Existing Foundations of a Major River Crossing Using Micropiles in Ontario, Canada," Proceedings of the 9th International Conference on Short and Medium Span Bridges, Calgary, AB, Canada.

Li, G. W., Pei, H. F., Yin, J. H., Lu, X. C. and Teng, J. (2014). "Monitoring and analysis of PHC pipe piles under hydraulic jacking using FBG sensing technology," Meas. J. Int. Meas. Confed., vol. 49, pp. 358–367

Ligozio, C.A., and Villalobos-Chapa, S. (2009). Evaluation of Milton Madison Bridge Substructures, a Report for Wilbur Smith and Associate. CTL Group Project No. 210727. Louisville, Kentucky.

Liu, Y. (1996) *Modeling the Time-to-Corrosion Cracking of the Cover Concrete in Chloride Contaminated Reinforced Concrete Structures*, Virginia Polytechnic Institute and State University. *Technical Report No. 54*,

Luh, Y.P. and Liu, Y.C. (2014). Bridge's Scour Monitoring System. Proceedings of the 2nd International Conference on Intelligent Technologies and Engineering Systems (ICITES2013), Lecture Notes in Electrical Engineering 293. Springer International Publishing Switzerland.

Maine DOT (2014). Bridge Design Guide, Maine Department of Transportation, Augusta, 2014.

Mao, J., Chen, J, Cui, L., Jin, W., Xu, C. and He, Y. (2015). Monitoring the Corrosion Process of Reinforced Concrete Using BOTDA and FBG Sensors. Sensors — Open Access Journal. Vol1 5(4):8866-83.

Mass DOT (2013). LRFD Bridge Manual, Massachusetts Department of Transportation, Boston

McManus, J.F., Phillip N. A., Stanton J.F., Turkiyyah, G. M. A. (1996). "Framework for the Constructability of Transportation Projects." Washington State Department of Transportation.

Mehta, P.K. and P.J.M. Monteiro (2014). *Concrete: Microstructure, Properties, and Materials*, 4th Ed., McGraw-Hill Education, New York, N.Y.

Melville B.W., Coleman S.E. (2000). Bridge scour. Highlands Ranch, USA: Water Resources Publications

Merzbacher, C.I., Kersey, A.D., and Friebele, E.J. (1996) Fiber optic sensors in concrete structures: a review. Smart Mater. Struct. Vol(5)196.

Modjeski and Masters, Inc. (2013). Widening of the Huey P. Long Bridge. Final Report to Louisiana Department of Transportation and Development. Baton Rouge, LA.

Molenar, K., Loulakis, M., Ferragut, T. (2014). Guide for the Process of Managing Risk on Rapid Renewal Projects

Moon, F., Romano, N., Masceri, D., Braley, J., Mertz, D.R., Samtani, N., Murphy, T., Lopez de Murphy, M. (2018) NCHRP Web-only Document 245: Bridge Superstructure Tolerance to Total and Differential Foundation Movements. *NCHRP Project 12-103*, Transportation Research Board, Washington, DC.

Moran, J. (2012). "The Fast 14 Project." Public Road Magazine, FHWA-HRT-12-004. Vol. 75 No. 6. Federal Highway Administration. Washington, DC

Mullins, A.G. and Kranc, S.C., Method for testing integrity of concrete shafts (2004). USF Patents. 762.

Mullins, G., Johnson, K., and Winters, D. (2007). Thermal Integrity Testing of Drilled Shafts. Florida Department of Transportation. Tallahassee, FL.

National Academy of Engineering (2017). "NAE Grand Challenges to Engineering in the 21st Century", National Academy of Sciences.

NCMA (2011). *TEK 18-1B* Evaluating the Compressive Strength of Masonry. National Concrete Masonry Association, Herndon, VA 20171

Neville, A. M. (1995). *Properties of concrete*. Pearson. Edinburgh Gate, Essex, England

New York City (2014) *New York City Building Code.* The New York City Department of Buildings.

NYCDOT (2014). Seismic Design Guideline for Bridges in Downstate Region. New York City Department of Transportation, available at https://www.dot.ny.gov/divisions/engineering/structures/repository/manuals/NYCDOT_SDGBDR_2016.pdf.

New York City Environmental Quality Review Technical Manual. (2014). NYC Mayor's office of Environmental coordination

New York State Department of Transportation. (2015). Geotechnical Control Procedure: Static Pile Load Test Manual. GCP-18. Revision #4. Geotechnical Engineering Bureau. Albany, NY.

Newnan D.G., Lavelle J.P. and Eschenbach T.G. (2013). Engineering Economic Analysis. Oxford University Press. 12th edition.

National Fire Protection Association (NFPA). NFPA 502 Standard for road tunnels, bridges, and other limited access highways. Quincy (MA): NFPA; 2008.

Nowak, A. S. (1999). NCHRP 368 *Calibration of LRFD Bridge design code* NCHRP Report No. 368, Transportation Research Board, Washington, D.C.

Olson Engineering (2014). Foundation Length Investigation Report, Parallel Seismic Testing, Route 1 Viaduct Bridge, Bath, Maine, Report to Haley & Aldrich, Inc., Portland, Maine.

Olson, L. D., Jalinoos, F., and Marwan, F. A. (1998), "Determination of unknown subsurface bridge foundations," Geotechnical Guideline No. 16, a summary of NCHRP 21-5 interim report, Federal Highway Administration, Washington, DC. Last accessed November 10, 2017.

Ontario Ministry of Transportation (MTO). (2011). *Provincial Highways Traffic Volumes 2011.* The Ontario Ministry of Transportation, Highway Standards Branch.

Ozbay, K., Jawad, D., Parker, N.A., Hussain, S. (2014), "Life cycle cost analysis State of the practice versus state of the art" *Transportation Research Record: Journal of the Transportation Research Board,* vol. 1864, pp. 62-70.

Pack, S.W., Jung, M.S., Song, H.W, Kim, S.H., and Ann, K.Y. (2010). Prediction of time dependent chloride transport in concrete structures exposed to a marine environment Cement and Concrete Research, Vol. 40, pp. 302–312

Paikowsky, S., Birgisson, B., McVay, M., Nguyen, T., Kuo, C., Baecher, G., Ayyub, B., Stenersen, K., O'Malley, K., Chernauskas, L., O'Neill, M. (2004). NCHRP REPORT 507, *Design (LRFD) for Deep Foundations.*, Transportation Research Board, Washington, D.C., 20001

Parola, A.C., Apelt, C.J, Jempson, M.A. (2000) *Debris Forces on Highway Bridges* NCHRP 445, Transportation Research Board, Washington DC, 20001

Portland Cement Association (PCA) (1994) "Assessing the Condition and Repair Alternatives of Fire-Exposed Concrete and Masonry Members", National Codes and Standards Council of the Concrete and Masonry Industries Skokie, Illinois, 60077

Potts, K., Ankrah, N. (2008). "Construction cost management, learning from case studies." Taylor Francis. New York.

Power, M., Fishman, K., Richards, R., Makdisi, F., Musser, S., Youd, T.L. (2004) *Seismic Retrofitting Manual For Highway Structures: Part 2 — Retaining Structures, Slopes, Tunnels, Culverts, and Roadways*, FHWA-HRT-05-067, Office of Infrastructure Research and Development, Federal Highway Administration (FHWA), McLean, VA 22101

Prendergast, L.J. and Gavin, K. (2014). A review of bridge scour monitoring techniques. Journal of Rock Mechanics and Geotechnical Engineering. Vol 6(2): 138–149.

Project Management Institute (2013). A guide to the project management body of knowledge: PMBOK. Fifth edition. Pennsylvania: Project Management Institute, Inc.

Rausche, F., Robinson, B., and Likins, G. (2004). On the Prediction of Long Term Pile Capacity from End-of-Driving Information. Contributions in Honor of George G. Gobel. American Society of Civil Engineers. Los Angeles, California.

Reel, R.S., and Conte, D.F. (1994). “Project Bridge Management in Ontario.” Transportation Research Circular 423, Transportation Research Board of the National Academies, Washington, DC; pp. 62–71.

Ryan, T.W., Mann, J. E., Chill, Z.M., Ott, B.T. (2012). “Bridge Inspector’s Reference Manual” *FHAW NHI 12-049*, Federal Highway Administration, Arlington, VA 22201

Saaty, T.L. (1980). “The Analytic Hierarchy Process.” McGraw-Hill, New York.

Saaty, T.L., and Vargus, L. (1982). “The logic of priorities.” Kluwer-Nijhoff Publishing, Boston.

Sabatini, P.J., Pass, D.G., Bachus, R.C. (1999). Geotechnical Engineering Circular (GEC) No. 4 *Ground Anchors and Anchored systems,* Report No. FHWA-IF-99-015, Federal Highway Administration (FHWA), Washington, D.C. 20590

Sabatini, P. J., Tanyu, B., Armour, T., Groneck, P., and Keeley, J. (2005). *Micropile Design and Construction*, FHWA NHI-05-039, National Highway Institute, U.S. Department of Transportation, Federal Highway Administration (FHWA), Washington, D.C. 20590.

Salem, S., and Miller, R. (2006). Accelerated Construction Decision-Making Process for Bridges. Midwest Regional University Transportation Center.

Samson, L. and Authier, J. (2011). Change in pile capacity with time: case histories. Canadian Geotechnical Journal, 1986, 23(2): 174-180, 10.1139/t86-027.

Samson, S., Reneke, J.A, and Wiecek, M.M. (2009). A review of different perspectives on uncertainty and risk and an alternative modeling paradigm. Reliability Engineering and System Safety. Vol. 94, pp. 558– 567

Samtani, N.C., and Kulicki, J.M. (2016). Incorporation of Foundation Deformations in AASHTO LRFD Bridge Design Process *SHRP 2 Solutions*, FHWA/AASHTO

Schaefer, V.R., Berg, R.R., Collin, J.G., Christopher, B.R., DiMaggio, J.A., Filz, G.M., Bruce, D.A., Ayala, D. (2016). Geotechnical Engineering Circular 13, *Ground Modification Methods – Reference Manual* FHWA-NHI-16-027, Federal Highway Administration (FHWA), Washington, D.C. 20590

Schaefer, V.R., Jalinoos, F. (2013). Characterization of Bridge Foundations Workshop Report, FHWA-HRT-13-101, Federal Highway Administration (FHWA), McLean, VA 22101

Schall, J.D., G.R. Price, G.A. Fisher, P.F. Lagasse, and E.V. Richardson, (1997a). NCHRP Report 397A *Sonar Scour Monitor— Installation, Operation, and Fabrication Manual*, Transportation Research Board, National Research Council, Washington, D.C.

Schall, J.D., G.R. Price, G.A. Fisher, P.F. Lagasse, and E.V. Richardson, (1997b) NCHRP Report 397B *Magnetic Sliding Collar Scour Monitor-Installation, Operation, and Fabrication Manual*, NCHRP Report No. 397B, Transportation Research Board, National Research Council, Washington, D.C.

SCEC, 1999, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Southern California Earthquake Center, University of Southern California, March, 63 pp.

Schiessl P., Raupach M. (1992). Monitoring System for the Corrosion Risk for Steel in Concrete Construction International; 14(7):52–55.

Sharp, S.R. (2016). "Use of Electrochemical Chloride Extraction and Associated Repairs to Extend the Beneficial Life of Reinforced Concrete Substructures." FHWA/VTRC 16-R16. Virginia Transportation Research Council. Charlottesville, VA.

Simon Wong Engineering. (2008). Final Seismic Strategy Report. North Torrey Pines Road Bridge at the San Diego Northern Railroad, Prepared for the City of Del Mar.

Smith. N.J., Merna, T. and Jobbling P. (2014). *Managing Risk in Construction Projects*, *3rd edition* John Wiley & Sons.

Sohanghpurwala, A. A. (2006). NCHRP 558 *Manual on service life of corrosion-damaged reinforced concrete bridge superstructure elements* NCHRP Report No. 558. Transportation Research Board, Washington DC

Sohn H, Farrar CR, Hemez F, Czarnecki J. (2004). A review of structural health monitoring literature: 1996-2001. Los Alamos National Laboratory, Los Alamos, USA

Song, H.W., Lee, C.H., and Ann K.Y. (2008). Factors influencing chloride transport in concrete structures exposed to marine environments Cement and Concrete Composites pp. 113–121.

Stark, T.D., Arellano, D., Horvath, J.S., Leshchinsky, D., (2004) NCHRP 529 *Guideline and Recommended Standard for Geofoam Applications in Highway Embankments* NCHRP Report No. 529. Transportation Research Board, Washington DC

Strauss, J., R. Nicholls, T. Chapman, and S. Anderson. (2007). "Drivers Affecting Frequency of Foundation Reuse and Relevance to U.S. Cities," GSP 158 Contemporary Issues in Deep Foundations, American Society of Civil Engineers, Reston, Va.

STRUCTUREPOINT LLC (2016) *spColumn User Manual,* Skokie, IL 60077

Thompson, P., Ford, K., Arman, M., Labi, S., Sinha, K., & Shirolé, A. (2012). NCHRP report 713: Estimating life expectancies of highway assets: Volume 1: Guidebook. *Transportation Research Board of the National Academies, Washington, DC,*

Tiberio, T. (2015). Reuse of Milton Madison Bridge Piers presentation to the 94th Annual Meeting of the Transportation Research Board, Washington, D.C.

Tsumoto, A., Kitiyodom P., Matsui, H., and Katsuzaki, Y. (2004). "Monitoring of load distribution of the piles of a bridge during and after construction" Soils Foundation, vol. 44, no. 4, pp. 109–117

TxDOT (2014) *Guidelines for the Use of Steel Piling for Bridge Foundations,* Texas Department of Transportation, Austin, TX 78701

U.S. Department of the Interior (1975) *Concrete Manual 8th edition*. Bureau of Reclamation, Washington DC 20402

Vassie, P. R. (1987). The Chloride concentration and resistivity of eight reinforced concrete bridge decks after 50 years of service. *TRRL RESEARCH REPORT*, (93).

Walls, J. III, Smith, M.R. (1998). Life-Cycle Cost Analysis in Pavement Design —Interim Technical Bulletin FHWA-SA-98-079, Federal Highway Administration, Washington, DC.

Wan, B., Foley, C.M., Ainge, S.W., Nguyen, C. (2013). Procedures, Cost and Effectiveness for Deteriorated Bridge Substructure Repair. *WHRP 0092-11-08* Wisconsin DOT, Milwaukee, WI, 53233

Watters, D.G., Bahr, A.J., Jayaweera, P. and Huestis, D.L. (2003). Smart Pebbles: Passive Embedded Wireless Sensors for Chloride Ingress Monitoring in Bridge Decks, SRI Report No. P11071 California Department of Transportation, Sacramento, CA.

Weyers, R. (1998). *Service life model for concrete structures in chloride laden environments*. Material Journal, Vol. 95, Issue 4, pp. 445-453. American Concrete Institute, Farmington Hills, MI 48331

Weyers, R.E., Prowell, B.D., Sprinkel, M.M., Vorster, M., (1993) "Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual" *SHRP-S-360*. National Research Council, Washington, DC.

Wightman, W. E, Jalinoos, F., Sirles, P., Hanna, K. (2003). Application of Geophysical Methods to Highway Related Problems, Federal Highway Administration, Central Federal Lands Highway Div., Publication No. FHWA-IF-04-021. Last accessed November 10, 2017.

Williamson, E.B., Bayrak, O., Williams, G.D., Davis, C.E., Marchand, K.A. McKay, A.E., Kulicki, J. and Wassef, W. (2010). *Blast Resistant Highway Bridges: Design and Detailing Guidelines.* NCHRP Report 645, Transportation Research Board, Washington, D.C.

Winch, G. (2002). Managing Construction Projects, an Information Processing Approach. Oxford: Blackwell Publishing.

Wright, W., Lattimer, B., Woodworth, M., Nahid, M., Sotelino, E. (2013) *Highway Bridge Fire Hazard Assessment* NCHRP Project 12-85. Transportation Research Board, Washington DC

Zaki, A., Chai, H.K., Aggelis, D.G, and Alver, N. (2015) Non-Destructive Evaluation for Corrosion Monitoring in Concrete: A Review and Capability of Acoustic Emission Technique. Sensors, 15(8), 19069-19101.

Zevenbergen, L.W., Arneson, L.A., Hunt, J.H., Miller, A.C. (2012) *Hydraulic Design of Safe Bridge* FHWA-HIF-12-018 HDS 7, Office of Bridge Technology, FHWA, Washington DC 20590

Zureick, A.H., Ellingwood, B.R., Nowak, A.S., Mertz, D.R., Triantafillou, T.C. (2010). *Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*. NCHRP 655 Washington, DC: The National Academies Press.

Zyka, K. and Mohajerani, A. (2016), “Composite Piles: A review”, Construction and Building Materials Volume 107, 15 March 2016, Pages 394–410.

Given the high percentage of deteriorated or obsolete bridges in the national bridge inventory, the reuse of bridge foundations is a viable option that can present a significant cost savings in bridge replacement and rehabilitation efforts. The potential time savings associated with foundation reuse can, in turn, reduce mobility impacts and increase the economic viability and sustainability of a project. However, existing foundations may have uncertain material properties, geometry, or details that impact the risks associated with reuse. Unlike a new foundation, an existing foundation may have been damaged, may not have sufficient capacity, and may have limited remaining service life due to deterioration.

Assessment of these issues as well as foundation strengthening and repair measures and innovative approaches to optimize loading are discussed in this report. To better demonstrate the engineering assessment of key integrity, durability and load carrying capacity issues, the report contains 15 case examples where foundation was reused by the owner agencies. On new construction, the report looks ahead and includes discussions on foundation design with consideration for reuse.

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