Evaluation of Constructed, Cast-in-Place (CIP) Piling Properties

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16. Abstract

Closed-end, round, cast-in-place (CIP) tubular friction piles are commonly used in bridge and retaining wall structures in the State of Wisconsin. Installation of CIP piles is typically performed by the contractor according to specified bearing capacities in the construction plans. These CIP piles have historically been installed at depths ranging from 30 ft. to 120 ft., with nominal diameters between 10-3/4 in. and 14 in. and shell thicknesses between 1/4 in. and 3/8 in.

In this study, a series of field-cast piles are investigated both experimentally and numerically to assess their structural capacity. The evaluation consisted of testing stub sections of pile with varying diameters (10-3/4 in., 12-3/4 in.), wall thicknesses (0.375 in., 0.5 in.) under various states of stress. The focus was on the axial capacity, the bond capacity and the compressive strength of the core material.

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Executive Summary

Closed-end, round, cast-in-place (CIP) tubular friction piles are commonly used in bridge and retaining wall structures in the State of Wisconsin. Installation of CIP piles is typically performed by the contractor according to specified bearing capacities in the construction plans. These CIP piles have historically been installed at depths ranging from 30 ft. to 120 ft., with nominal diameters between 10-3/4 in. and 14 in. and shell thicknesses between 1/4 in. and 1/2 in. Upon driving the tubular piles to capacity, concrete is placed by allowing it to free-fall within the tubular, with care taken to minimize intermittent voids.

Generally these piles are designed on a resistance based method which is a function of the soil properties the pile is driven through. However, there is a structural capacity that can be considered to be the upper bound on the loading of the member. WisDOT practices employ a conservative approach in the design of these CIP pilings by neglecting the contribution of the steel shell and reducing the design compressive strength of the concrete core. These adjustments are due to uncertainties in the shell integrity after long-term environmental exposure and the in-place properties of the core including compressive strength and interface composite action. The focus of this investigation is to evaluate the behavior of CIP pilings with a specific focus on the actual compressive strength of the placed concrete and the composite action between the concrete core and steel shell. The previously described aspects of long-term integrity are not included in this study.

The overall objective of this project is to characterize the axial capacity of typical CIP tubular piles used by WisDOT in bridge and retaining wall structures. Of primary interest is the characterization of the actual compressive strength of the in-place concrete due to uncertainties in the placement method. The other key objective includes an investigation of the level of composite action between the concrete core and the steel tubular shell.

In this study, a series of field-cast piles were investigated both experimentally and numerically to assess their structural capacity. The evaluation consisted of testing

stub sections of pile with varying sizes (10-3/4” dia., 12-3/4” dia.), wall thicknesses (0.375”, 0.5”) under various states of stress.

For the experimental program, several full-length piles were partially driven and filled in the field to represent as near in-situ conditions as possible. These full-length piles were cut up into smaller testable sizes, which were also believed to be representative of the short braced lengths for piles driven into the ground. The smaller pile stub sections were run through a set of different tests, with a representative sampling taken from different depths of each pile for each test. The tests included compression tests on the entire stub cross-section, compression tests of the stub section where only the core was loaded, flexural tests of short pile sections, push-through tests, and testing of cored out samples from stub sections. In addition to the experimental program, a numerical study (finite element) was performed on the axially loaded sections to allow for extrapolation of the behavior of the piles to conditions more representative to in-service conditions.

Results from the investigation suggest that the current design approach for nominal axial capacity is conservative when compared to test results, in some cases up to 300% reserve capacity was observed. The results also indicate that the methods used for placing provide a sound concrete core with adequate bond capacity and consolidation for the intended application.

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1 Introduction

1.1 Foundations

The foundation is one of the most important elements of a structure because it serves as the component that anchors the structure to the earth. A failure in the foundation could lead to a catastrophic failure of the entire structure. This is one reason many substructure elements are over-designed and have a high factor of safety (Coduto 2001). However, with the advancement of construction technology, bridges are spanning longer distances and as a result larger foundation elements are necessary along with a better understanding of their behavior to optimize designs and material usage. In general, there are two main categories of foundations, shallow and deep foundations. An overview of both foundation types is presented in the following subsection.

1.1.1 Shallow Foundations

Shallow foundations are typical for structures with smaller loads and when the soil beneath the structure can easily support the weight of the structure, such as a house with a block wall foundation. The main two types of shallow foundations are spread footing foundations and mat foundations (Coduto 2001).

A spread footing transfers the loads from a single column, group of columns, or wall down to an enlarged bottom, which distributes the load over a much larger bearing area. These are by far the most common type of foundation due to their low price and ease of construction. Spread footings can be built in many different shapes, with the most common shapes being square, rectangular, circular, continuous, combined, and ring.

The mat foundation is actually very similar to the spread footing, but instead of supporting only one column or wall, the entire structure has a single “mat” beneath the entire footprint of the structure. Typically, mat foundations are used when loads are high enough or the soil is poor enough as to make the spread footings cover about 50 percent or more of the structure’s footprint area. They are also used when there is high potential for differential settlement due to soil conditions, erratic loading, and non-

uniformly distributed lateral loads. Mat foundations are also practical when the uplift loads are larger than what a spread foundation could support, and when the foundation is below the groundwater table, so waterproofing is a large concern (Coduto 2001). While shallow foundations are often used in bridges, the focus of this study is on deep foundations, and no further discussion on shallow foundations will be presented.

1.1.2 Deep Foundations

Deep foundations are used when standard spread footings are not capable of supporting the structure. Deep foundations transfer the loads down to soils layers below the main structure that are capable of supporting the loads of the structure (Coduto 2001). The main types of deep foundations are piles, drilled shafts, and caissons. Other types of deep foundations include mandrel-driven thin shells filled with concrete, auger-cast piles, pressure injected footings, and anchors. Pile foundations are typically prefabricated members that are driven or forced into the ground. Drilled shafts are made by drilling a cylindrical hole in the ground, adding reinforcing steel and filling the hole with concrete. Caissons are prefabricated like piles, however, they are much larger. Generally caissons are boxes or cylinders that are sunk into the ground to the desired depth by digging around them, and then filling them with either concrete or soil (Coduto 2001). While all of these deep foundation types are relevant to bridge construction, this investigation focuses on the application of piles foundations, specifically cast-in-place (CIP) tubular friction piles; more detailed discussions on the other deep foundations is readily available in literature.

1.1.2.1 Pile Foundations

Deep foundations (e.g. piles or drilled shafts) are commonly used in bridge and retaining wail structures in the State of Wisconsin when competent bearing soil is not present near the ground surface. The primary function of these deep foundations is to 1) transmit the load of the structure through the poor bearing soil to soil with adequate bearing capacity, 2) eliminate objectionable settlement; 3) transfer load from a structure through erodible soil (in scour environments); 4) anchor structure against uplift; and 5) resist lateral loads from earth pressures and external forces (WisDOT 2011). Deep

foundations have included timber piles, cast-in-place piles (CIP), reinforced piles, precast piles, and H-piles. While most of these piles are commonly used, the focus of this study is on a subset of cast-in-place piles, which are closed-end, round, cast-in-place (CIP) tubular friction piles. These piles have historically been installed in Wisconsin at depths ranging from 30 to 120 ft., with diameters between 10-3/4 – 14 in. and shell thicknesses ranging between 1/4 and 1/2 in.

Concrete-filled steel tubular members, when designed correctly, can take advantage of both component strengths and behave as a single unit. The shell, typically the steel element, is then filled with concrete. The steel shell serves as formwork for the concrete and also as the longitudinal reinforcement. Locating steel at the farthest point from the centroid maximizes the moment of inertia in the cross section, which then increases the flexural capacity. Also, the steel shell provides confinement for the concrete similar to spiral reinforcement in columns, which can increase the compressive strength of the concrete (Baig *et al.* 2006). Lastly the presence of the concrete core fills the space in the steel tube, enhancing the buckling capacity of the steel shell, while the presence of the steel shell helps to prevent spalling of the concrete core.

Concrete-filled steel tubular sections are used for many different construction elements, ranging from foundation elements to columns. In bridge foundations, concrete-filled steel tubulars behave as a continuously braced column due to the support provided by the surrounding soil with the capacity defined as the lower of the structural capacity of the pile and the bearing (geotechnical) resistance of the supporting soil, with the later typically controlling and the former serving as an upper bound on the load that can be safely applied if the pile was driven to refusal in a solid bed with a capacity greater than that of the pile. In many structures the maximum structural capacity can be achieved by driving piles into high strength layers. These piles are often classified as displacement-type piles with the majority of the resistance provided by the shaft resistance (Figure 1). Design provisions for concrete-filled steel tubular piles are included within the AASHTO LRFD Bridge Design Specifications

(AASHTO 2010) with many state agencies adopting their own modifications based on past performance and uncertainty of down-hole conditions.

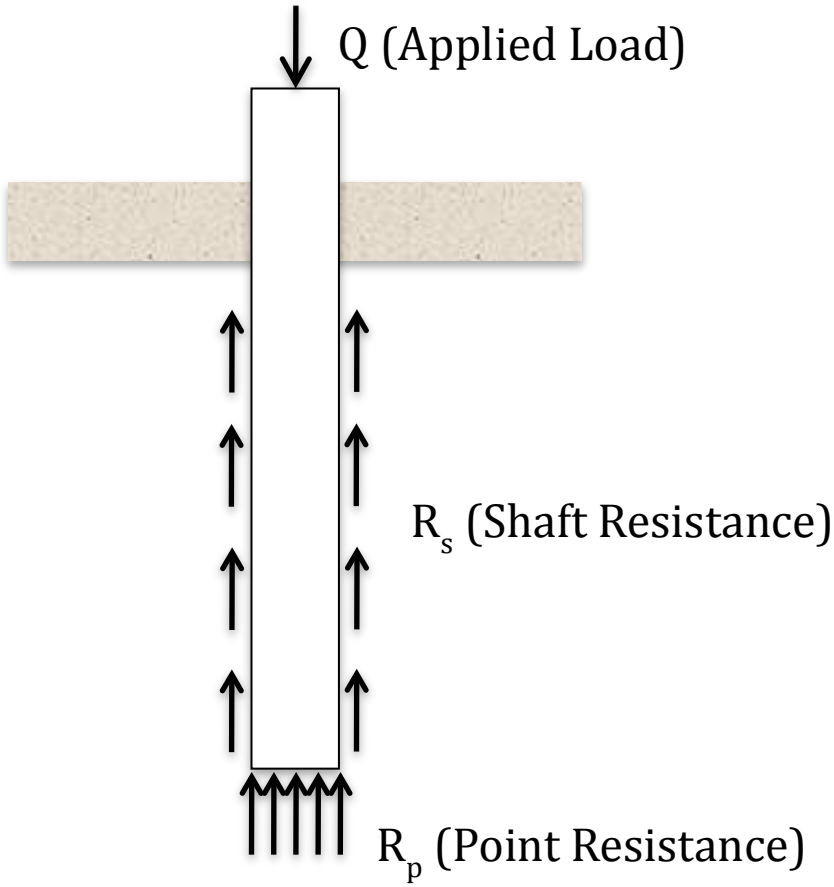


Figure 1 - Resistance Distribution for Axially-Loaded Pile (Adopted from WisDOT 2011)

1.1.3 AASHTO LRFD Practice

1.1.3.1 Structural Resistance

AASHTO LRFD (AASHTO 2010) provides two alternatives to determine the capacity of a concrete-filled steel tubular pile. The first approach is a simplification that considers the concrete core, while the second approach considers composite action between the steel and concrete, and evaluates the member as a fully bonded system.

The AASHTO LRFD sets out the two methods quite explicitly and specifies when each can be used. For composite action to be considered, the area of steel of the

shell must be greater than or equal to four percent of the entire cross-section. The case of non-composite action considers the partial contribution of the concrete core and any steel (rebar, prestressing) inside this concrete in determining nominal capacity, P_n. For spiral reinforced non-composite and standard tie reinforced piles, the nominal capacity would be calculated using Equation 1 and Equation 2, respectively.

$$P_n = 0.85\left[0.85 f_c^{'}\left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right)\right] \qquad \text{Equation 1}$$

$$P_n = 0.80\left[0.85 f_c^{'}\left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right)\right] \qquad \text{Equation 2}$$

where:

P_n = nominal axial resistance without flexure (kips)

A_g = gross area of concrete pile section (in^2)

A_{st} = total area of longitudinal reinforcement (in^2)

A_{ps} = total area of longitudinal prestressing reinforcement (in^2)

f_{ps} = effective stress in prestressing tendons after losses (ksi)

f_y = specified yield strength of reinforcement (ksi)

$f_c^{'}$ = concrete compressive strength (ksi)

ε_{cu} = ultimate usable strain in concrete under compression

E_p = elastic modulus of prestressing tendons (ksi)

When composite action can be considered, the nominal capacity is based on elastic and inelastic compression member behavior using an effective section of steel and concrete. This method is similar to the composite section behavior within the AISC Steel Construction Manual (2005). The composite section nominal capacity is calculated using the relationships in Equation 3.

$$P_n = \begin{cases} 0.66^{\lambda} F_e A_s & if \quad \lambda \le 2.25 \\ \dfrac{0.88 F_e A_s}{\lambda} & if \quad \lambda > 2.25 \end{cases}$$ Equation 3

where:

$$\lambda = \left(\frac{Kl}{r_s \pi} \right)^2 \frac{F_e}{E_e}$$

$$F_e = F_y + C_1 F_{yr} \frac{A_r}{A_s} + C_2 f_c^{'} \frac{A_c}{A_s}$$

$$E_e = E \left[1 + \frac{C_3 A_c}{n A_s} \right]$$

K = effective length factor

l = unbraced length

r_s = radius of gyration of the steel section

A_s = cross sectional area of steel shell

A_c = cross sectional area of concrete core

A_r = cross sectional area of steel reinforcement

F_y = yield strength of steel shell

F_{yr} = yield strength of longitudinal reinforcement

$f_c^{'}$ = concrete compressive strength

E = modulus of elasticity of steel shell

C_1, C_2, C_3 = composite column constants (see Table 1)

Table 1: Composite Section Constants

	Filled Tubes	Encased Tubes
C_1	1.00	0.70
C_2	0.85	0.60

C_3	0.40	0.20

1.1.3.2 Bearing (Geotechnical) Resistance

The AASHTO LRFD Bridge Design Specification recommends that the axial resistance be verified during pile installation or using a static analysis if dynamic methods are determined to be unsuitable. Verification during installation allows for use of load tests, dynamic tests, wave formulae or dynamic formula whereas the static analysis considers the summation of the pile tip and side resistances that are based on soil types. A more detailed overview of the methods for determining the bearing resistance of piles is presented in the AASHTO LRFD Bridge Design Specification Section 10.7.3.8 (AASHTO 2010), but is beyond the scope of this study.

1.1.4 WisDOT Practices

1.1.4.1 Structural Resistance

Current WisDOT design practice (WisDOT 2011) for concrete-filled steel tubular piles is based on the same principles as those included within the AASHTO LRFD Bridge Design Specifications (AASHTO 2010), with modifications to account for uncertainties in the steel shell integrity over time, in-place compressive strength of the concrete core, and degree of composite action between the concrete core and steel shell. AASHTO LRFD treats pilings as compression members, with the design procedure determined by the type of member (e.g. concrete compression member – section 5.7.4.4 or composite member – section 6.9.5.1). WisDOT practice allows for a similar treatment, but due to the potential for corrosion and uncertainty in the level of composite action, the contribution of the steel tubular is neglected. This treatment reduces the behavior to that of a concrete compression member [Equation 4]. In addition, the allowable concrete design compressive strength is limited to 3.5 ksi based on uncertainties in the placement practice, construction difficulties, and historical inconsistency with the plans. These modifications result in a significant reduction in

design capacity when compared to the AASHTO LRFD methodology and in turn require more piles to achieve the desired capacity.

$$P_n = 0.80\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{Equation 4}$$

where:

P_n = nominal axial resistance without flexure (kips)

A_g = gross area of concrete pile section (in^2)

A_{st} = total area of longitudinal reinforcement (in^2)

f_y = specified yield strength of reinforcement (ksi)

$f_c^{'}$ = concrete compressive strength (ksi)

1.1.4.2 Bearing (Geotechnical) Resistance

The WisDOT Bridge Design Manual (WisDOT 2011) provides an equation for the bearing resistance (Equation 5) that is based on the AASHTO LRFD Bridge Design Specification method for static analysis, but does not include recommendations for in-situ verification. Also included in the WisDOT manual are tables with ranges of nominal shaft resistance typical to the State of Wisconsin based on soil type (Table 2 and Table 3).

$$R_n = \varphi_{sat} R_p + \varphi_{sat} R_s \qquad \text{Equation 5}$$

where:

R_n = Nominal resistance

φ_{sat} = Resistance factor for driven pile, static analysis method

R_p = Nominal point resistance of pile (tons)

R_s = Nominal shaft resistance of pile (tons)

Table 2 - Average Shaft Friction Values for Cohesive Materials (WisDOT 2011)

Soil Type	q_u (tsf)	Nominal Shaft Resistance (psf)
Very soft clay	0 to 0.25	---
Soft clay	0.25 to 0.5	200 to 450
Medium clay	0.5 to 1.0	450 to 800
Stiff clay	1.0 to 2.0	800 to 1,500
Very stiff clay	2.0 to 4.0	1,500 to 2,500
Hard clay	4.0	2,500 to 3,500
Silt	---	100 to 400
Silty clay	---	400 to 700
Sandy clay	---	400 to 700
Sandy silt	---	600 to 1,000
Dense silty clay	---	900 to 1,500

Table 3 – Average Shaft Friction Values for Granular Materials (WisDOT 2011)

Soil Type	N_{160}	Nominal Shaft Resistance (psf)
Very loose sand and silt or clay	0 to 6	50 to 150
Medium sand and silt or clay	6 to 30	400 to 600
Dense sand and silt or clay	30 to 50	600 to 800
Very dense sand and silt or clay	over 50	800 to 1,000
Very loose sand	0 to 4	700 to 1,700
Loose sand	4 to 10	700 to 1,700
Firm sand	10 to 30	700 to 1,700
Dense sand	30 to 50	700 to 1,700
Very dense sand	over 50	700 to 1,700
Sand and gravel	---	1,000 to 3,000
Gravel	---	1,500 to 3,500

1.1.5 State Specific Design Codes

While new bridge designs use the AASHTO LRFD, states also maintain their own design codes, many of which are based on current and historical AASHTO design standards. A survey and summary of available design approaches is presented in this section.

The design approaches adopted by most states can be categorized as composite, non-composite, state-specific or no explicit information. The composite design approaches used are similar to that of the AASHTO LFRD and are used by Indiana, Maine, Massachusetts, Nebraska, Nevada, and South Carolina. The non-composite design approaches consider only the concrete or reinforced concrete core and are used by Florida, Idaho, Missouri, Montana, New Jersey, Pennsylvania and Wisconsin. However, Idaho and Montana use a non-composite section with only the steel contributing to the capacity, while Missouri allows the steel shell to count as reinforcement in the section and New Jersey refers to a previous version of the AASHTO, which also uses a non-composite section.

Some states have their own methodology as to how to design or use piles. Some of these include tabulated capacities of specified pile sizes and types, allowable stresses or even prohibit the use of pipe piles. Alabama, Connecticut, Delaware, Minnesota, North Carolina, Texas, Virginia all use tables to give the engineers the values for the capacity of their standard size piles, while Michigan, Ohio, Oregon, Rhode Island, and Washington D.C. only design piles by the resistance based method. California requires allowable stress design and Iowa does not allow for the use of pipe piles at all. A summary of the available state methodologies is presented in Table 4.

Table 4 - Available State/District Transportation Agency Methodologies

Design Method	States
Composite	Indiana, Maine, Massachusetts, Nebraska, Nevada, and South Carolina
Non-Composite	Florida, Idaho, Missouri, Montana, New Jersey, Pennsylvania and Wisconsin
Tables	Alabama, Connecticut, Delaware, Minnesota, North Carolina, Texas, Virginia
Resistance Based	Michigan, Ohio, Oregon, Rhode Island, Washington D.C.
Other	California (allowable stress), Iowa (pipe piles not allowed)
Not Listed	Alaska, Arkansas, Arizona, Colorado, Georgia, Hawaii, Illinois, Kentucky, Maryland, New Hampshire, New Mexico, New York, South Dakota, Tennessee, Vermont, Washington, West Virginia, Wyoming

1.1.6 Other Approaches

Additional methods for determining the structural capacity of concrete-filled steel tubular members include those within the Manual for Steel Construction (AISC 2005) and the Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 2008); however, these methods are essentially the same as those within the AASHTO LRFD Bridge Design Specification. Additionally, these provisions are primarily intended for building design and not directly applicable to bridge infrastructure and are not discussed further.

1.2 Study Objectives

The overall objective of this project is to characterize the axial capacity of typical CIP tubular piles used by WisDOT in bridge and retaining wall structures. Of primary interest to this goal is the characterization of the actual compressive strength of the in-place concrete due to uncertainties in the placement method. The other key objective includes an investigation of the level of composite action between the concrete core and the steel tubular shell.

Piles were evaluated in multiple configurations to determine how they behave under different boundary and loading scenarios. The major considerations in this

project were the structural capacity of the pile under axial loading, and whether the piles act as a composite member, or whether there is slip between the materials in the pile, making each material act independently. In addition, the in-place compressive strength of the core concrete was evaluated. Results from this investigation provide the information necessary for the evaluation of current design practices pertaining to CIP tubular piles. The information developed as part of this project should allow WisDOT to determine whether the current assumptions are overly conservative or if there is a potential economic benefit to a modified design approach.

1.3 Report Organization

This report examines the behavior of concrete-filled steel tubular members used as foundation elements in bridge infrastructure. This project investigates the structural capacity of cast-in-place steel tubular piles. Included in this report are results and discussion of a numerical and experimental investigation of the behavior of concrete-filled steel tubular piles. The report is organized according to the following chapters: 1) Chapter 2: Literature Review, 2) Chapter 3: Means and Methods, 3) Chapter 4: Analysis, 4) Chapter 5: Findings and Recommendations, and 5) Chapter 6: References. Additional data from the experimental program is included in the appendices.

2 Literature Review

The use of driven piles for deep foundations has been around for many years. These piles originally were made of timber, but over the years have evolved into concrete and steel or combinations of the two materials. With the ever increasing load requirements and the need for high bearing capacity, concrete-filled steel pipe piles were introduced around 1925 (WisDOT 2011). As highlighted in the previous chapter, these piles can be described as hollow steel tubes that are driven into the ground and then filled with concrete. The resulting composite member takes advantage of some of the optimum characteristics of the two materials, compressive strength of the concrete core and braced (internally) compressive strength of the thin steel shell (Lam and Wong 2005). These piles are also in the category of slender members, but the lateral restraint provided by the surrounding soil medium results in a system behavior more similar to a very short (stub) column where lateral buckling does not occur.

2.1 Previous Research

Included in this section is a summary of literature pertinent to the performance of cast-in-place steel tubular piles. While limited studies related to the structural capacity of concrete filled steel tubular piles were found, pertinent studies on circular concrete-filled tubulars (CFTs) have been included. The literature review included herein is not considered to be an exhaustive summary of literature, but rather synthesis of key aspects related to the behavior of concrete filled steel tubular piles including both structural capacity and interface bond capacity. No additional consideration on the bearing capacity is presented, as this is beyond the scope of the investigation.

2.1.1 Structural Capacity

As highlighted in the previous chapter, the structural capacity of a cast-in-place steel tubular pile can be considered equivalent to a composite column, but with different loading and boundary conditions. The effects of these loading and boundary conditions are what make the behavior of the pile complex as is evident from the high degree of conservativeness used in their design (e.g. neglecting steel shell, reducing the concrete

strength, etc.). However, it is the structural capacity that ensures that the pile will resist the applied loads and transmit them to the surrounding soil without failure.

A literature survey on cast-in-place steel tubular piles yielded little results with the exception of resistance-based design capacity (Olson and Shantz 2004; Gilbert *et al.* 2005; Xiao *et al.* 2005; Jung *et al.* 2006; Kim *et al.* 2009; Long *et al.* 2009). This research on the resistance-based method dealt with drivability and how different soil types reacted under different loads and applications, and as such was deemed irrelevant for this research project. However, extensive studies have been performed on concrete-filled tubular members (CFT), which are more common in building infrastructure. CFTs are piles with longer unbraced lengths that are installed rather than driven. The lateral restraint provided by the soil is such that the full length of the pile may be upwards of forty feet, but the unbraced length may be as little as a few feet or the pile may even be continuously braced (or unbraced lengths approaching zero), all based on the stiffness of the soil into which the pile is driven. CFTs, on the contrary, have unbraced lengths on the order of twelve to thirty feet, depending on story height of the building in which they are used. In addition, most CFTs are HSS (hollow structural shapes) with D/t (diameter to wall thickness) ratios between 10 and 70 whereas, concrete filled tubular piles D/t ratios range between 20 and 40 and can be either straight or spiral welded as the seams.

2.1.1.1 CFT Experimental Studies

Most experimental studies on CFTs have been performed on stub sections due to testing challenges associated with full-length sections. These shorter test sections would be expected to be more representative of the behavior of in-service piles, where unbraced lengths are short due to the restraint provided by the surrounding soil. Much of the research evaluated the behavior of different shapes (circular and rectangular) and sizes (diameter and wall thickness) of the steel shells, along with variations in concrete core compressive strength.

Many of the different studies varied the cross section and material properties of the CFTs. In a study by Yamamoto (2000), CFTs with three different diameters (4”,

8.5”, 12.5”), three different side dimensions (3.9”, 7.8”, 11.8”), and four different concrete strengths (4, 5, 7, 10 ksi) were tested to failure. Three different loading conditions were used in the test. The sections were either loaded completely, only the concrete core was loaded, or only the hollow steel shell was loaded. From the strains obtained during testing, it was found that strains would increase until local buckling started, at this point, the strains would essentially plateau, while the steel shell was buckling. This study developed recommendations for a new design equation able to predict capacities with no more than ten percent error. In another study by O’Shea (2000), three different loading methods for CFTs were investigated. The sections were loaded using the same three methods as in the study by Yamamoto (2000). From these different loading patterns, it was determined that, when only the steel shell is loaded, the core does not contribute to change the capacity. When loading only the core or the entire section, the core significantly adds to the capacity.

Fujimoto *et al.* (2004) studied how the ratio of the diameter to wall thickness or side length to wall thickness affected eccentrically loaded CFTs. The test sections had three steel strengths (29, 85, 110 ksi), three concrete strengths (3, 6, 12 ksi), and various ratios of diameter to shell thickness (~16 to 150). A constant concentric load was applied followed by an increasing eccentric load; however the concentric load was decreased such that the total load remained constant. Fujimoto found that the CFTs exhibited a very stable moment-curvature relationship, but also warned that mixing high strength concrete with lower strength steel may yield lower overall strengths than using both high strength steel and high strength concrete. In another study by Gupta (2007), the effects of the chemical composition of concrete, and different additives and admixtures, on the strength of different sized CFTs, was examined. Differing amounts of fly ash, two industrial additives, and steel tubes of three different sizes (2”, 3”, 4”) were used. The capacities obtained from testing were greater than the original theoretical capacities, due to the lack of considerations of the confinement effects. When compared to models proposed by other researchers (Giakoumelis and Lam 2004) which included the effects of confinement, the results were better represented. In a study by Lam and Wong (2005), CFTs with different grade steel shells were

investigated. The authors examined how a rectangular stainless steel shell would affect the capacity. A total of three different concrete strengths (4, 9, 14 ksi), and varying wall thicknesses (0.1”-0.2”) were tested. Their study determined that the stainless steel shells did not provide the same level of confinement as normal steel shells. Schneider (1998) investigated the axial capacity of concrete-filled steel tubes with diameter-to-wall thickness ratios between 17 and 50 and length-to-diameter ratios between 4 and 5. Results from Schneider’s study revealed that significant confinement effects are not achieved until that axial load exceeds 90% of the yield strength of the member and that axial ductility for the circular columns investigated can exceed 10.

Several of these studies also examined how the capacities predicted by the design methods compared with experimental results. Baig et al. (2006) studied the capacity of CFTs compared with the theoretical capacities from several different design methods. The size and wall thickness of the pipe sections varied greatly, with only the steel strength (36 ksi) and concrete strength (4ksi) remaining constant. The authors compared the test results with the theoretical design methods of the Eurocode-4, ACI (American Concrete Institute), the Australian Standard Codes, AISC (American Institute of Steel Construction) LRFD (load and resistance factored design), and the Chinese Code. During this testing, it was noted that when localized and member buckling occurred, the concrete assumed the shape of any steel deformations. The capacity predicted by Eurocode-4 was found to underestimate the actual capacity by 14-21%, the ACI and Australian Codes also underestimate the capacity by 16-32%, the AISC LRFD underestimated the capacity by 10-40%, while the Chinese Code overestimated by 7-30%. In a study by Lam and Wong (2005) using concrete-filled stainless steel shells, the Eurocode-4 overestimated the capacity while the ACI and Australian codes underestimated the capacity. Results from a study by Schneider (1998) revealed that the AISC design equations (AISC 2005), which are similar to those of the AASHTO LRFD (AASHTO 2010) provided a reasonable and conservative estimate of the axial capacity of concrete-filled steel tubulars; however, this trend decreased for larger diameter tubulars. In this work, specimens were significantly smaller than the piles used by WisDOT, ~50% smaller or more, and were both circular and rectangular in

shape. Tests by Sakino *et al.* (2004) on concrete-filled steel tube stub sections resulted in capacities greater than predictions of the nominal squash load, equivalent to the AASHTO LRFD non-composite capacity. This finding was attributed to the beneficial effects of confinement that are achieved at later stages of loading as the Poisson's ratio of concrete approaches that of steel. Overall, these experimental works suggest that design codes generally provide a reasonable approximation of capacity.

2.1.1.2 CFT Numerical Studies

Some studies also looked at how to better model the behavior of CFT sections. Hu (2003) studied how to model confining pressure in ABAQUS using test data from previous research as a baseline. When creating the model, it was determined that both a confining pressure and an increased concrete strength were needed. The confining pressure is due to the fact that the two materials have different Poisson ratios, causing them to expand at different rates. The model also needed to take the confined concrete strength into consideration because the presence of confining pressure holds the concrete back from spalling. After these elements were included in the model it was found to agree with the test data. In a study by Schneider (1998) a finite element model developed in ABAQUS, without consideration of confinement effects, was able to accurately mimic the results of capacity tests of stub sections.

Other research examined potential design methods for CFTs. In a study by Xiao (2010), confined CFTs were examined. Confined CFTs are normal CFTs with additional transverse reinforcement at locations with local buckling potential. This additional reinforcement was thought to delay any local buckling seen in normal CFTs, and could be easily retrofitted onto existing columns. The confined CFTs exhibited improved capacity up until rupture of the wrap. Even after rupture, the capacity of the wrapped CFT was higher than that of a normal CFT until high strain values were observed. Gilbert *et al.* (2005) studied incorporating a lower bound into the capacity of the LRFD method. It was found that offshore applications were using a lower bound and had much lower failure rates. This was thought to be caused by a physical limit to the

smallest load a pile can support, which is greater than zero. After some testing, the authors suggested a few methods to incorporate the lower bounds into future designs.

2.1.2 Interface Bond Capacity

The bond between the concrete core and the interior surface of the steel shell in cast-in-place steel tubular piles is critical to the performance of the composite system. Without this bond, the system can be assumed to behave as a non-composite column with a reduced stiffness as compared to the composite counterpart. For cast-in-place steel tubular piles, the performance of this bond in not well understood as a result of construction practices related to placing techniques (e.g. free falling concrete vs. tremie) and long-term condition of the pile due to factors related to environmental exposure. Researchers have extensively studied the bond of steel to concrete and concluded that the bond capacity is highly dependent on material characteristics, surface conditions, and state of stress, all of which have a high degree of uncertainty at the concrete core/steel shell interface in cast-in-place steel tubular piles. However, limited studies are available on the bond behavior of cast-in-place steel tubular piles. In an analytical study by Johansson and Gylltoft (2002), the interface bond was shown not to significantly influence the axial capacity when the entire cross-section is loaded, but influenced the failure buckling mode when the load was applied only to the core section. On the contrary, Fam *et al.* (2004) observed an increase in the capacity of stub sections with unbonded cores as compared to composite stub sections, which was attributed to the enhanced confining effects in the non-composite section. In a study by Roeder *et al.* (1999) on the bond strength in CFTs, the influence of concrete strength on bond was shown to have no significant impact whereas tubular diameter and diameter to shell thickness ratio showed some correlation, with bond strength decreasing with an increase in both.

2.1.3 Summary of Literature Review

A literature review on the in-place behavior of cast-in-place steel tubular piles provided limited studies on the capacity of these members aside from studies on the bearing capacity. However, this investigation did produce a number of studies on the

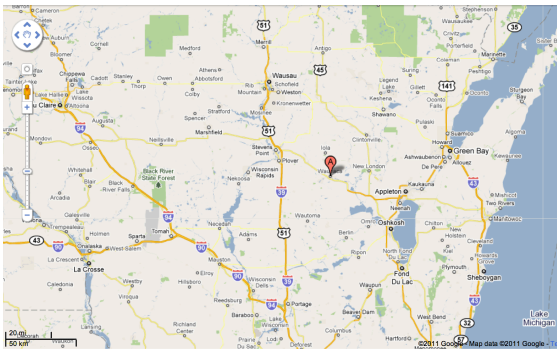
behavior of concrete-filled steel tubulars, which are becoming more common in the building industry. These members have similar attributes to the cast-in-place steel tubular piles except that these members are typically smaller in diameter and supported differently, maintaining longer unbraced lengths; however the studies on structural capacity of these members are pertinent to this investigation. The literature review included a summary of the key investigations on these members including results from both compression behavior and bond behavior. A general trend that can be derived from these studies is that the capacity of the tubular is highly dependent on loading scenario and influenced by the diameter to shell thickness ratio. These studies also highlighted the potential influence on concrete core strength and unbraced length which are considerations in this study.

3 Means and Methods

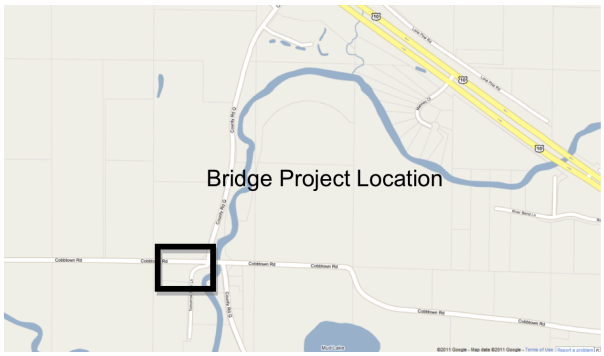
The objective of this study was to evaluate the capacity and behavior of concrete-filled steel tubular piles. As part of this investigation, a series of laboratory experiments were performed on pile sections that were field cast. For the laboratory investigation, several full-length piles were field-cast in conditions as similar as possible to in service conditions and later cut into smaller sections that could be tested. The emphasis of the experimental investigation was on the structural capacity of the piles as well as the interface bond capacity between the concrete core and steel shell. The tests performed included composite section loading, non-composite section loading, cored sample testing, flexural testing of short pile sections, and a test on the bond strength between the core and shell. In addition to the experimental study, a numerical investigation was also conducted to simulate the response of alternative scenarios that were not captured in the experimental investigation. Included in this section is a summary of the pile fabrication, cutting, and testing program along with the details of the finite element model development used in the numerical investigation.

3.1 Pile Fabrication

The piles used in this study were constructed in conjunction with an ongoing new bridge construction project near the intersection of Cobbtown Road and County Road Q, just to the northwest of Waupaca, Wisconsin (Figure 2). The project was managed by Pheifer Brothers Construction and pile construction was performed as to not interfere with the existing project.



(a)



(b)

Figure 2 – (a) General Location – Waupaca, WI; (b) Specific Site Location (Google 2011)

This site was chosen for its location away from any major highway and the dates available for driving. The piles used for the project were circular closed end steel pipe piles, which were driven and filled simultaneously as the bridge piers for this new bridge. Four sample piles were installed in a row next to the bridge site. These piles were later removed and used for this research project. Two of the piles are 10-3/4 in. diameter piles, one with 3/8 in. wall (pile 1) and one with a 1/2 in. wall (pile 2). The other two piles are 12-3/4 in. diameter piles (piles 3 and 4), both have a wall thickness of 3/8 in (Figure 3). Based on the proposed testing program, the pile shells used were thicker than expected, resulting in greater capacities than planned; however, WisDOT standards (WisDOT 2005) allow for the contractor to use larger piles than specified for drivability purposes. All four of the piles were approximately forty feet long and satisfied testing requirements for ASTM 252 Grade 2 or 3 (See Appendix A – Chemical Testing Results of Steel Shells). The 12-3/4 in. diameter piles were spiral formed, whereas the 10-3/4 in. diameter piles were cold rolled and seam welded.

Figure 3 - The four piles and the caissons

3.1.1 Driving the Piles

Driving of the piles occurred in parallel with a new bridge construction near Waupaca, Wisconsin. The pile driver (Figure 4) used on this project was a single piston Delmag diesel hammer. It was chosen based on the sandy soil conditions of the site. The pile driver had a single head that could install many different diameters of circular piles (10-3/4 in., 12-3/4 in., and 14 in.), as shown in Figure 5. This head allowed the installation to go much smoother and faster when changing the size of circular piles on job sites and reduced the need for multiple drive heads when dealing with the steel tubular piles.

Figure 4 - Pile driver

Figure 5 - Pile drive head

The driving of the piles proceeded quickly, due to soil conditions in the area. The piles were only partially driven (~15 ft.), to allow for ease of installation and removal. If the piles were fully driven, much more work would be required to remove the piles as the piles get their strength from the frictional resistance between the side of the pile and the soil. Fifteen feet was chosen because the soil directly beneath the pile was capable of supporting the pile vertically and a filled caisson placed around the pile provided enough lateral support to counter any horizontal loads the pile would experience.

Because the piles were not driven to full depth, they were insulated to mimic in-situ condition to which typical piles are exposed. Several alternatives to insulate the portion of the pile above ground were investigated. One idea was to build a set of walls

around the piles and backfill the voids between the piles and the wall with soil. This was found to be too costly, due to the required number of supports and bracing in the walls necessary to resist the soil pressure. Another idea was to use a full size concrete caisson, typically used for bridge piers, to surround the piles and fill the caisson with soil. This is similar to the wall idea, but no longer required the construction of walls and bracing. Finally, it was determined that the best solution was to use individual caissons that are a few inches larger than the piles and then fill the void between the pile and the caisson with soil. This would require much less time and cure as similar to the piles that were fully driven into soil while being more economical.

The selected caissons were a steel shell that was open on both ends and was several inches in diameter larger than the pile it was going to be placed around. The four caissons were lifted placed around each one of the piles. These caissons can be seen in Figure 6, which shows the caissons placed around the piles before they were filled with soil from the site, and before the piles were filled with concrete.

Figure 6 - Unfilled piles with the caisson placed around them

3.1.2 Casting the Piles

The concrete for the piles was placed using a pump truck which takes the concrete from the concrete truck and pumps it through a hose. The hose was lowered slightly into the piles for the pour (Figure 7). The piles were filled by allowing the concrete to simply fall, which is standard practice for the contractor. To prevent a cavity from forming at the top of the section, the hose was slowly removed while still pumping.

Several standard on-site tests were performed on the wet concrete during casting to ensure it was within specifications. The slump of the mix was 2-¾ in., and the entrained air content was 5%, both of these were within WisDOT specifications (2005), however the final mix details were not available for inclusion in this report. Compression test results performed as part of this project are included in the results section. Along with these standard tests, a total of 20 four by eight inch cylinders were cast for testing later. Nine of these cylinders were transported back with the researchers after casting and the remaining eleven were left wrapped by soil from the site to cure in a similar manner to the piles. The nine that were transported back were used to run preliminary compression tests to verify that the concrete in the piles had sufficient capacity for extraction. These cylinders were used primarily for this purpose and as a gauge for future tests, but are not included in the results as the transportation was outside of ASTM standard practice.

Figure 7 - Concrete pour into piles

3.1.3 Curing the Piles

The area between the caissons and the piles was filled with soil from the area to help reasonably simulate underground temperature and moisture conditions. Since the caissons are far too high to reach by hand, a concrete bucket was used to fill the void between the piles and the caissons (Figure 8).

Figure 8 - Caissons being filled with soil

With soil filling the voids between the pile and the caisson, the piles were left to cure before removal. Cylinders were tested at five and seven days to ensure that the concrete strength was high enough for the piles to be safely removed and stored. The five-day average strength was 3,915 psi, while the seven-day average strength was 4,349 psi. This was determined to be sufficient strength, and the following day, the piles were extracted. After the piles were removed, they were set aside for pickup (Figure 9) and later transported to the Michigan Tech campus for testing.

Figure 9 - Piles after being pulled

3.1.4 Cutting the Piles

Different methods for cutting the full-length piles down to testable section sizes were investigated. One method was a large hydraulic shear, as it would be quick and could be done on site. However the finished surface of the piles after cutting would be deformed and crushing of the concrete was expected. Another method was water jetting, with a computer-aided cutting machine, which leaves a very smooth and clean surface. However there were no local businesses capable of handling such large samples on their water jet. Alternatively, a company would have to use a mobile unit, which is not as easily controlled and cannot guarantee an even cut each time. The chosen method was a diamond wire saw. These saws are used to remove large sections of buildings, sever underwater pipelines, cut large bricks of rock in quarries, and many other heavy-duty tasks. This method was found to leave a flat, clean surface after cutting, very similar to that of ones cut by a stationary water jet. Another advantage to the wire saw over the water jet is the portability of the wire saw. These systems are made to be used on site and can be adapted for use in many different configurations. It was determined that a wire saw typically used to cut underwater pipe would be best suited for this situation. Cutting Edge Services Corp. provided the diamond wire saw equipment and services for the pile cutting operations.

To support the piles during the cutting, stands were needed. The support was constructed using 8x8 timbers, which were of minimal cost, and would easily support

the load. The original plan was to use four two-foot sections for each support, but during construction, this was modified to three two-foot sections for each support, allowing more supports to be built while still easily supporting the load, see Figure 10. To help hold the piles on the support and prevent them from sliding, some 2x4s were cut at a forty-five degree angle and screwed down to the top of the timber, giving lateral support by acting as wedges.

Figure 10 - Pile supports for cutting

The saw that was used to cut the piles can be seen in Figure 11 and Figure 12. Once the saw was clamped onto the section, it averaged about twenty minutes per cut. The diamond wire itself was cooled with water and the water was also used as a lubricant to help cut through the material and keep dust down. There were a total of eighty-two cuts for the four piles.

Figure 11 - Diamond wire saw

Figure 12 - Diamond wire saw cutting

The piles were cut in the Michigan Tech Benedict Laboratory into 12 in., 18 in., and 11 ft. sections, with the specimen size based on the intended testing plan. These tests include: full section compression, section core compression, cored samples, flexural testing, and push-through testing, and can be found in more detail in the next section. The sections were numbered according to which pile they were cut from, and then lettered in the order they were removed, with A being at the top and Z being at the

bottom. The numbering nomenclature and intended test scheme for each pile is shown in Figure 13.

The test specimens selected for compression testing were 18” long, as this size fit into the test frame. The stub sections dimensions were selected to mimic the short unbraced length of the continuously supported piles in the ground. Specimen lengths of 18 in. and 12 in. were selected to extract core samples at various locations along the piles. These sizes were chosen, as it would allow for 4 in. by 8 in. cylinders to be cored out, while fully using the most of each pile length. The eleven-foot long sections were reserved for flexural testing which allowed for a ten-foot clear span to be tested. Six inches on either side was determined to be adequate for the supports on the load frame at Michigan Tech. Out of each pile section, a total of one 11 ft. section, two 12 in. sections, and fifteen to eighteen 18 in. sections were cut.

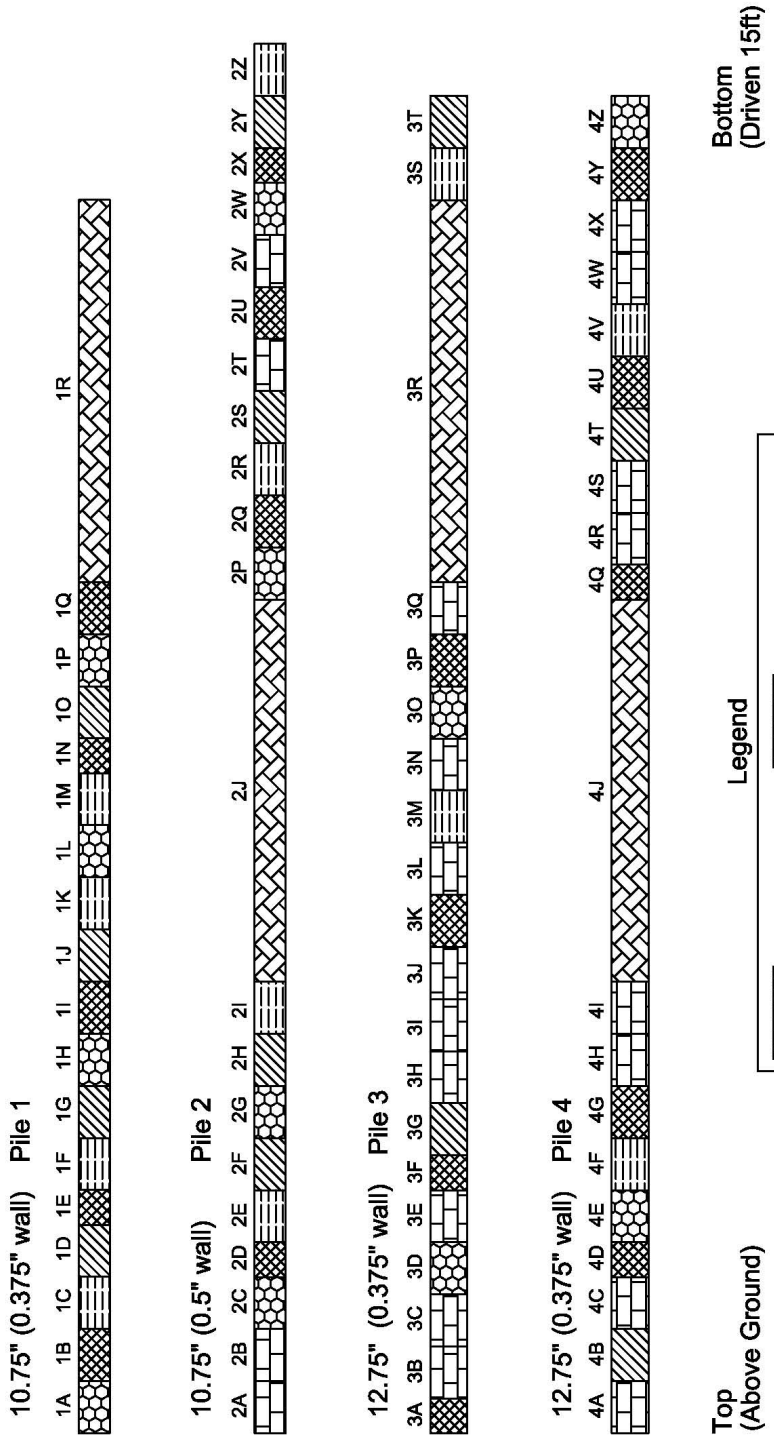


Figure 13 – Cut Pile Nomenclature

3.2 Testing the Piles

This section describes the full section compression (composite loading), section core compression (non-composite loading), cored samples, push-through (bond), and flexural (bond) testing performed on the stub sections of each pile, along with the machines and instrumentation used for the different testing.

3.2.1 Testing Machines

The load frame used for the compression testing was a MTS 315.03 with the loading applied through a fixed head and rigid loading base pair. This frame is capable of exerting a total load of 1 million pounds and has a total stroke of 4 in (Figure 14). A MTS technician calibrated the frame before the testing was started. The software that runs the frame was set up in displacement control, at a rate of 0.01 in. per minute. This load rate was calculated to give a test of a standard length and to provide a smooth data set.

Figure 14 - MTS 315.03 Load Frame (Benedict Laboratory – Michigan Tech)

The other testing machine, a Baldwin 300CT (Figure 15) with the loading applied through a fixed base and rigid loading head pair, was used for the testing of the core samples and cylinders and has a capacity of 300,000 pounds. The machine is set to

the ASTM Standard C31 for testing cylinders, which gives a loading rate of 35 psi per second.

Figure 15 - Baldwin Cylinder Test Machine (Benedict Laboratory – Michigan Tech)

For the testing of the flexural specimens, a self-reacting load frame with two 55-kip MTS actuators was used (Figure 16). There was no standard test procedure available for the flexural testing, but the equipment is capable of cyclic or static loading under a variety of configurations. For this testing, the actuators were used in conjunction to provide a monotonic loading condition.

Figure 16 – Self-reacting frame (Dillman Structural Engineering Laboratory – Michigan Tech)

3.2.2 Data Acquisition and Instrumentation

The axial compression and flexural testing used a Campbell Scientific CR9000X data acquisition system (DAQ), which is considered a modular measurement and control system. The DAQ has numerous input capabilities, but for this project only linear variable differential transformers (LVDT) and strain gauges.

The strain gauges were all from Vishay Micro-measurements. For the axial compression and flexural testing the strain gauges used were 1/8 in. long, 120 Ω series BT strain gauges (EA-06-125BT-120).

The other instrumentation used was a load cell and an LVDT. The LVDT measured total distance traveled by the loading head during testing, while the load cell measured the applied load. These measurements were used to find the stress in the samples and the amount of deflection the specimen experienced. The displacement from the LVDT was also used to verify that the frame was loading at the specified displacement control rate.

3.2.2.1 Axial Compression Testing

The first test conducted on the pile sections was the axial compression tests. The compression tests consisted of an axial compression of the entire cross section, a confined axial compression test on the concrete core, axial compression testing on each individual cored concrete core sample, and a push-through test, see Figure 13 for specimen location details.

3.2.2.1.1 Full Section Compression Tests

The full section compression test was used to investigate the ultimate capacity of the pile stub (18 in. nominal length) sections (Figure 17). Four pile sections from each size configuration were tested. To obtain a better representative value of the true average capacity, 4 sections were taken from different areas of each pile. Each pile stub section had two strain gauges attached onto their surface. One strain gauge was placed to measure the longitudinal strain at the mid-height of the pile stub section and the other was placed to measure the tangential strain at the mid-height. The second

strain gauge captured any outward deformation from Poisson's effects of the steel shell at the mid-height.

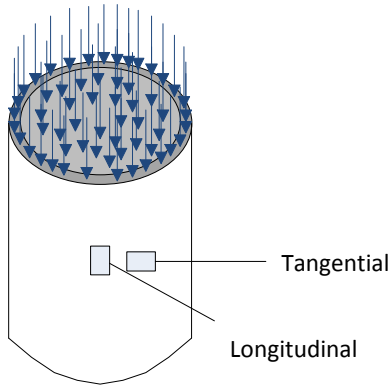


Figure 17 - Full section Loads

The test specimens were placed into the load frame with a T-100 (100,000psi yield) plate steel cap at either end. This cap was used to protect the loading head of the test frame and to provide a flat surface for loading (Figure 18). All of the specimens were loaded in compression to evaluate the capacity of the section.

Figure 18 - Full section loading setup

3.2.2.1.2 Section Core Compression Tests

The section core compression test was used to determine the capacity of the section (18 in. nominal length) if loaded only through the core section (Figure 19). The

resultant capacity was expected to be some representation of the confined compressive strength of the concrete core.

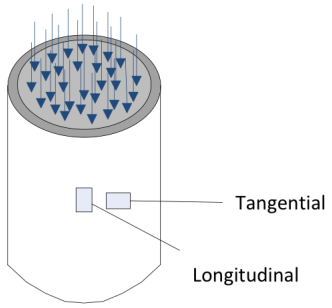


Figure 19 - Section core loads

The setup for this test was very similar to the whole section compression test except the Grade 60 plates were used to load only the core section rather than the entire cross section. One inch thick plates were cut to fit just inside the pile's steel shell with enough clearance as to not contact the steel shell on both ends of the specimen (Figure 20). Each pile stub section had two strain gauges attached onto their surface. One strain gauge was placed to measure the longitudinal strain at the mid-height of the pile stub section and the other was placed to measure the tangential strain at the mid-height. The second strain gauge captured any outward deformation from Poisson's effects of the steel shell at the mid-height. These gauges were intended to observe the behavior of the steel shell when the concrete core was loaded. Ideally, this was expected to capture the failure of the concrete core, along with any expansion in the steel shell. All of the specimens were loaded in compression to evaluate the capacity of the section.

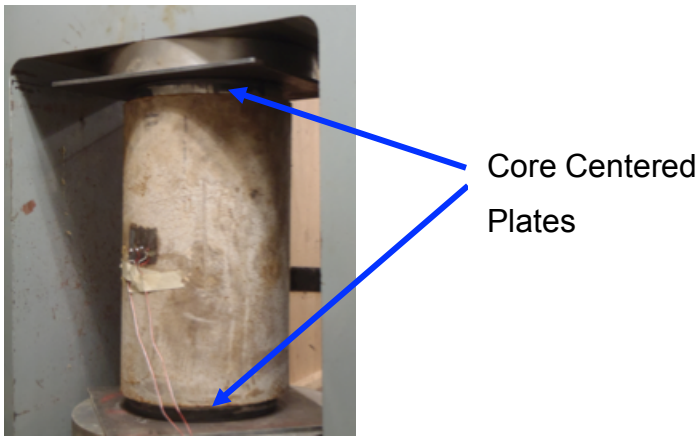


Figure 20 - Section core loading setup

3.2.2.2 Cored Sample Tests

Core samples were taken from various locations in the pile sections to determine the representative concrete strength throughout the pile; the sections used for this can be seen in Figure 13. From the sets of piles with the same outside diameter, at least one section was taken out of every five foot length of the pile. Eight of these sections were 12 in. tall and ten sections were 18 in. tall specimens. A drill press with a four by thirteen inch core bit was used to core samples out of the pile sections (Figure 21).

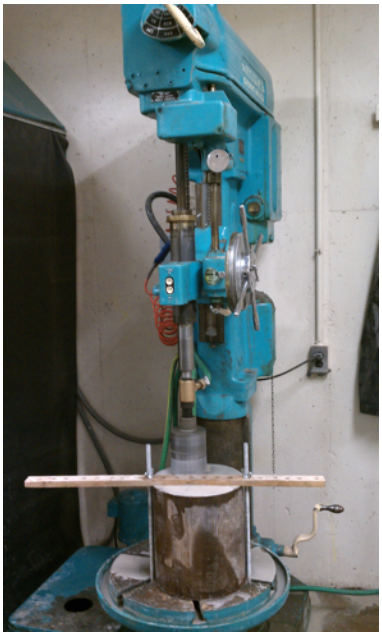


Figure 21 - Drill press setup

The maximum numbers of cores were taken from each pile section. All cores were drilled out at the same rate and at a depth of ten inches, so that when the core was removed, it would still allow for enough material to be end ground to yield a 4 by 8 in. cylinder. These cylinders were then each individually tested in the Baldwin 300CT compression machine mentioned above following ASTM C-31 (2003).

3.2.2.3 Push-through Tests

The final compression testing conducted was a series of push-through tests. Used to determine the bond strength between the concrete core and the outer steel shell, these tests were conducted using the MTS million pound test frame. Each test section had one strain gauge to record the vertical strain in the steel shell at the midpoint. This strain gauge and the displacements were used to capture the load under which the concrete core began to move in relation to the shell. At that point, the bond strength has been overcome and only friction is holding the sections together (Figure 22).

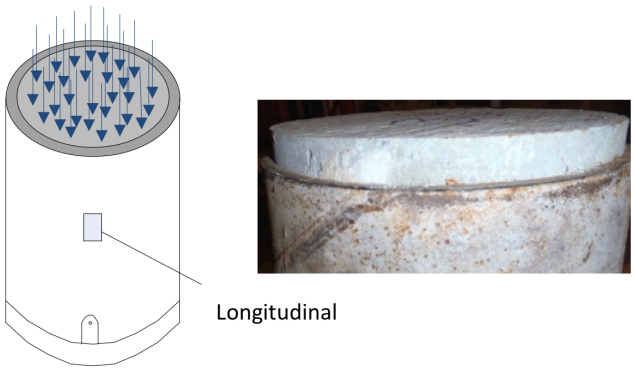


Figure 22 - Push-through loads

For this setup, two rings were constructed using 1 in. thick grade 60 steel. These rings were cut so the outside had a diameter of fourteen inches to match the loading platens on the loading frame. The inside diameter was cut so there was enough clearance to allow the concrete core to be pushed through but also enough overlap over the steel to hold the shell back. Two rings had to be constructed to accommodate the diameters of test sections (Figure 23). The platens used for the confined concrete

testing were also used in this test so that only the concrete would be loaded. This allowed for the core to be forced out of the shell, see Figure 24.

Figure 23 - Push-through rings

Figure 24 - Push-through setup

3.2.2.4 Flexural Testing

Three-point flexural testing was performed on the four 11 ft. long pile sections (Figure 13). This testing configuration was selected to evaluate the bond performance of the pile sections under flexural conditions. The intent was to evaluate if slip occurred along the core/shell interface prior to failure of the piles. The tested sections were taken from different locations in the pile with one taken from the above ground curing regime and one below for each of the pile diameters. For testing purposes, each of the piles had a steel plate welded near the ends to prevent rolling and allow the specimens to rest flat during testing. Under these plates a roller was placed on one end and a pin

style connection on the other, providing a clear span of 10 ft. for testing. The loading was applied at the center of the 10 ft. clear span, such that each support was 5 ft. from the center of the section. Strain gauges were mounted around the perimeter at midspan. A general schematic of the loading and strain gauge configuration is shown in Figure 25 and Figure 26.

Figure 25 – Experimental setup for flexural testing

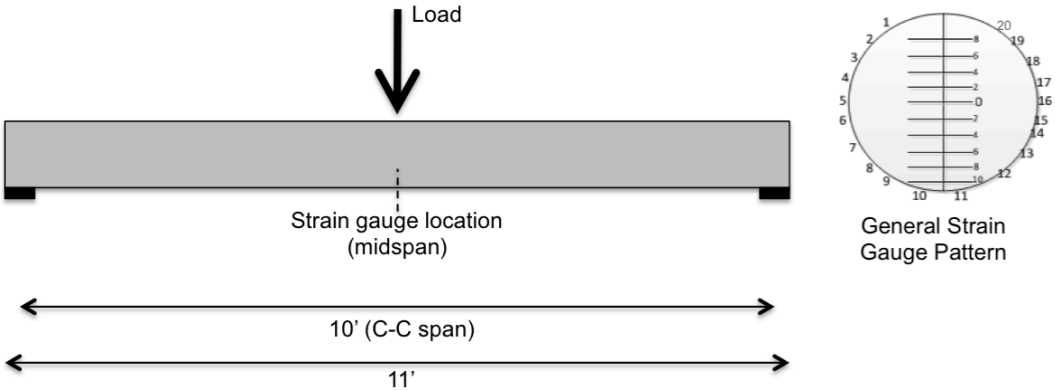


Figure 26 – Flexural Testing Configuration

For the first test (12-3/4 in. diameter pile) there were a total of 20 strain gauges mounted around the circumference of the pile section. These gauges were positioned every 2 in. apart with one being located at the center on the bottom and progressing up

every 2 in. on both sides. This configuration was employed only on the first specimen to capture the full response and aid in gauge positioning for the subsequent tests. For the remaining tests, the number of strain gauges was decreased to a total of 9 gauges to minimize labor because the additional gauges on the first test did not provide much value to the analysis. These 9 gauges were also equally spaced throughout the circumference of the sections. On the 10-3/4" dia. specimens, the gauges were located with one again at center on the bottom and then two more gauges were located at 3 in. spacing on each side of the center gauge, after the 2 gauges in each direction there was a 5 in. gap to the next gauge followed by one last gauge on each side that is 3 in. above. A similar pattern was used for the last 12-3/4" dia. specimen, but an adjustment was made to account for the larger diameter (3.5 in. spacing vs. 3 in. and 5.75 in. spacing vs. 5 in.). In addition to the strain gauges there was a series of digital LDVTs (Mititoyo) mounted on the ends (top and bottom) of the specimens to measure potential slip between the concrete core and steel shell during testing. Digital Mititoyo gauges were used on the first test to determine if any slip occurred on the sections. These gauges did not register any slip during the load testing. As such there was deemed no slip in the sections and in the subsequent tests the Mititoyo gauges were swapped out for several LVDTs to measure the true displacement. In addition to the external instrumentation, load and displacement were recorded from the actuator at the cross-head location.

3.3 Finite Element Model

A finite element model was created to simulate the behavior of the pile sections used in the experimental study. These models were limited to the axial compression scenarios described in the previous subsections and do not include models of the push through (bond) or flexural configurations. The commercial finite element software package ANSYS 13.0 was used to develop the models and all models were created using 8-node solid elements (Solid 92). This model was later expanded to assess the in-service behavior of the piles in varying soil conditions in the State of Wisconsin.

Initially a preliminary model was created for validation of the modeling approach, which consisted of a simple test case of a uniaxial load applied to a homogeneous section using Hooke's relationships for stress and strain (Vable 2002). The model was created for both a generic concrete (Figure 27) and steel (Figure 28) cross-sections and compared with Hooke's relationships (Equation 6- Equation 8). The validation models used a modulus of elasticity of 3,800 ksi and a Poisson's ratio of 0.15 for the concrete. Similarly, a modulus of elasticity of 29,000 ksi and a Poisson's ratio of 0.30 was used for the steel model. As expected the basic models were able to simulate the theoretical behavior within the linear elastic range and were deemed acceptable for this study.

$$\varepsilon_x = \frac{1}{E}\left[\sigma_x - \nu\left(\sigma_y + \sigma_z\right)\right] \qquad \text{Equation 6}$$

$$\varepsilon_y = \frac{1}{E}\left[\sigma_y - \nu\left(\sigma_x + \sigma_z\right)\right] \qquad \text{Equation 7}$$

$$\varepsilon_z = \frac{1}{E}\left[\sigma_z - \nu\left(\sigma_x + \sigma_y\right)\right] \qquad \text{Equation 8}$$

where:

$E = Elastic\ Modulus$

$\upsilon = Poisson\ Ratio$

$\sigma_x = stress\ in\ the\ radial\ direction$

$\sigma_y = stress\ in\ the\ tangential\ direction$

$\sigma_z = stress\ in\ the\ longitudinal\ direction$

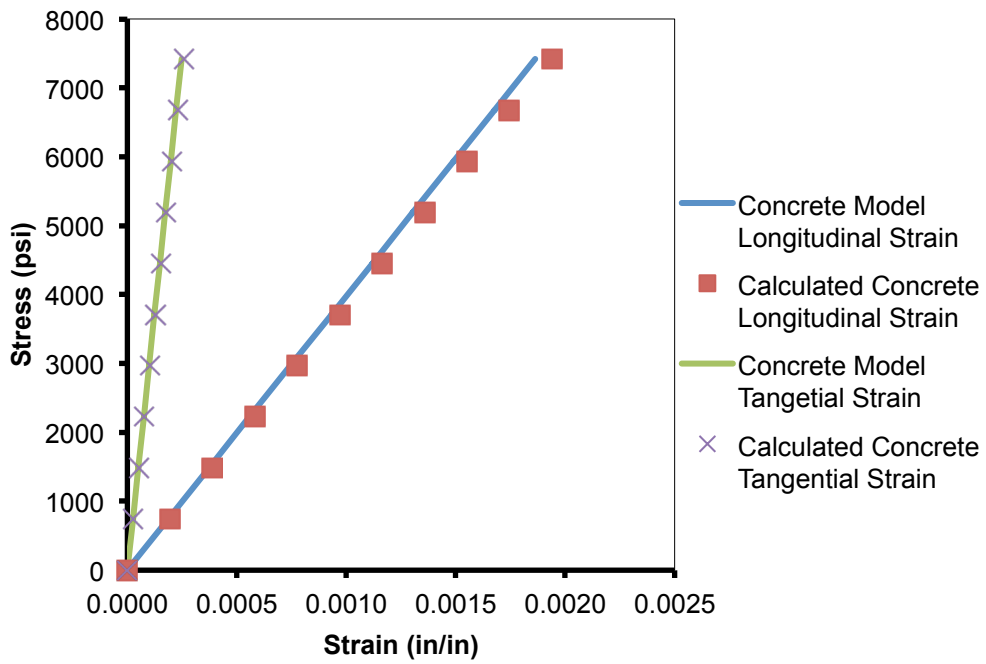


Figure 27 – Concrete Model Verification

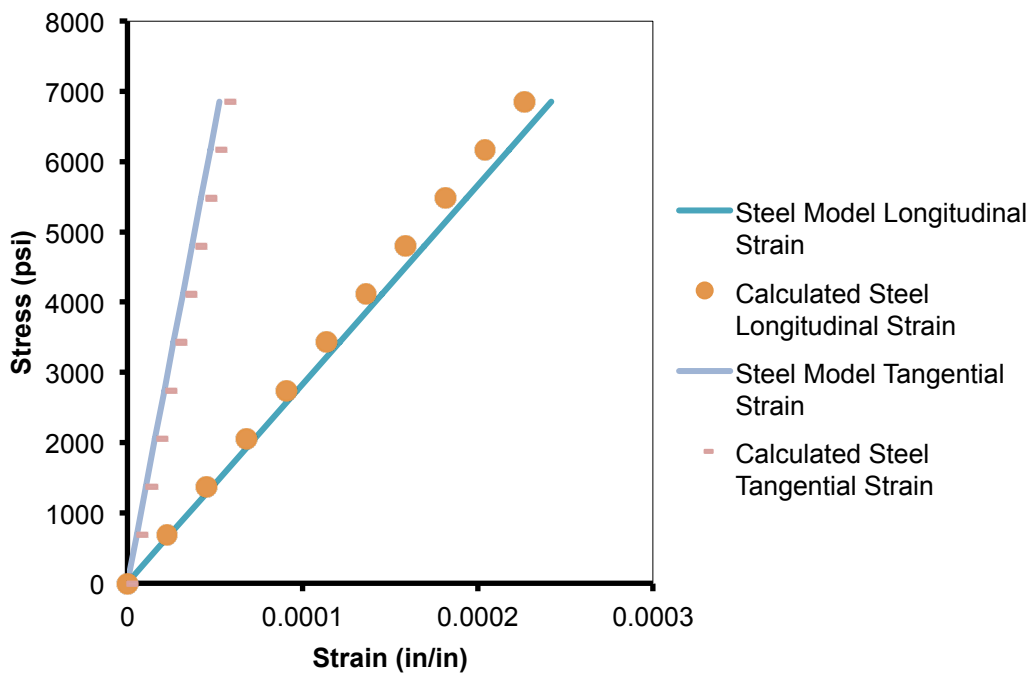


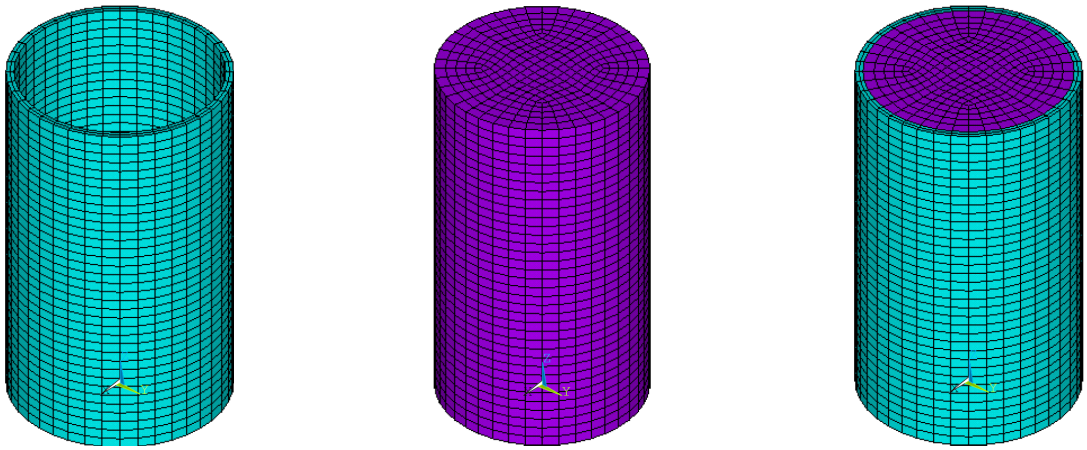
Figure 28 – Steel Model Verification

Following the initial model validation, additional finite element models were developed for the stub sections to verify the results obtained from experimental investigations. The models developed consisted of 18 in. length sections having 10-3/4 in. and 12-3/4 in. diameters, which matched those from the experimental program. A summary of the complete geometrical properties of the simulated pile sections is presented in Table 5.

Table 5 - Geometry of the pile stub sections simulated by finite element analysis

Pile No.	Diameter (in.)	Wall Thickness (in.)	Core Radius (in.)
Pile 1	10.75	0.375	5.000
Pile 2	10.75	0.500	4.875
Piles 3 & 4	12.75	0.375	6.000

To ensure the bonding condition between the concrete core and the steel wall, the corresponding nodes from both materials were merged together at the interface. After the volumes were modeled, the material properties were added to the element. The general modeling configuration of the stub-sections in the three dimensional environment is depicted in Figure 29 for the piles with different geometrical properties.



(a) Steel external shell (b) Concrete core (c) Composite section

Figure 29 - 3D modeling of pile stub-section

Linear elastic material models were chosen for the concrete and steel. For the steel external shell, an elastic modulus of 29,000 ksi and the Poisson's ratio of 0.3 were

chosen for the simulation. Moreover, for the concrete core, the elastic modulus of 3.8, 4.4, and 5.1 ksi were used with the Poisson's ratio of 0.15. These modulus values were derived using the relationships between elastic modulus and compressive strength (Equation 9 and Equation 10) within ACI 318 and AASHTO LRFD, respectively. The compressive strengths used in the relationships were 4.5, 5.85, and 8 ksi, which were selected based on the variability of the core test results (*see section 4.1*).

$$E_C = 57{,}000\sqrt{f'_C} \qquad \text{Equation 9}$$

$$E_C = 33{,}000 K_1 w_c^{1.5}\sqrt{f'_C} \qquad \text{Equation 10}$$

where:

$E_c = Elastic\ modulus\ of\ concrete$

$K_1 = correction\ factor\ for\ aggregate\ type\ (taken\ as\ 1.0)$

$f_c' = compressive\ strength\ of\ concrete$

$w_c = unit\ weight\ of\ concrete$

It should be noted that none of the stub sections were tested to failure because the experimental load frame was unable to provide sufficient load to capture the failure. Moreover, the ratio between the length of the pile sections and their diameter is small enough to expect the pile sections to demonstrate fully plastic response under the compressive stresses, rather than an Euler buckling (global buckling) mode of the failure. On the other hand, the local buckling phenomenon was neglected in the analysis due to the lack of corresponding experimental data for validation purposes, because the capacity of the test machine was incapable of initiating a local buckling failure of the steel shell. As a result all of the numerical models were limited to the linear elastic range.