

Experimental and analytical investigation of bridge timber piles under eccentric loads

Daniel J. Borello^{a,1}, Bassem Andrawes^{a,*}, Jerome F. Hajjar^{a,1}, Scott M. Olson^{a,1}, James Hansen^{b,2,3}

^a Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana, IL, USA

^b Shannon & Wilson, Inc., Seattle, WA, USA

ARTICLE INFO

Article history:

Received 30 July 2009

Received in revised form

8 February 2010

Accepted 25 March 2010

Available online 24 April 2010

Keywords:

Bridges

Timber piles

Eccentric load

Nonlinear analysis

ABSTRACT

This paper examines the structural behavior of bridge timber piles under eccentric compression loading. Samples of the piles were retrieved from a recently collapsed bridge and experimentally tested under compression and combined compression and flexure. The experimental timber pile response was used to calibrate a numerical model of full timber piles of a prototype bridge, including material and geometric nonlinearity as well as soil–structure interaction. The numerical results illustrated that the pile strength was significantly reduced under eccentric load compared to concentric load. Therefore, it was concluded that the effect of compression–flexure interaction on bridge timber piles must be checked during bridge design and/or rating, especially in the case of simply supported superstructures where loading on one span may lead to eccentric loading on a timber pile group.

© 2010 Elsevier Ltd. All rights reserved.

1. Introduction

Timber piles represent an economical foundation system that has commonly been used for small bridges since the middle of the 18th century [1]. They are typically sawn from Southern Pine, Douglas fir, Oak, Red Pine or Cedar [2]. Similar to the natural properties of a tree, they are usually round with a natural taper that increases frictional capacity but still must meet certain criteria for quality, straightness and diameter [3]. Under ideal conditions they have an indefinite service life [4]. Above the water level they are treated with a preservative (e.g., creosote) to prevent decay, which can be effective for at least 40 years [5]. According to current code provisions, timber piles are designed and/or rated primarily for concentric compression loading, assuming that the superstructure provides sufficient rigidity to prevent eccentricities in the piles [6]. However, this assumption is invalid if the superstructure consists of simply supported spans. In this common case, if only one bridge

span carries live load (Fig. 2), the reactions from two adjacent spans could be different and the compressive force transmitted to the piles would be eccentric. The resulting interaction of compression and flexural loads in the piles could negatively impact timber pile strength, and therefore, this loading case should be evaluated in timber pile design. The primary goal of this paper is to shed the light on the effects of applying eccentric loads on the strength of round timber piles. Experimental testing of deteriorated and relatively new piles retrieved from a 32 year old bridge was conducted and the results were used in a numerical analysis of full-span piles under eccentric loads.

2. Previous work

The Army and Navy in 1924 first investigated the interaction of combined compression and flexure loads in wood for airplane construction [7]. Experimental testing of 51 mm (2 in.) square specimens and analysis based on Euler buckling illustrated that the material capacity was a function of load eccentricity as well as the slenderness of the section. Interaction equations were later developed to describe the test results [8] as a function of the axial stress and the flexural stress. These equations were shown to provide reasonable accuracy for short, intermediate and long beam-columns.

In 1982, Zahn [9] tested Western Hemlock specimens measuring 51 mm (2 in.) × 152 mm (6 in.) nominal under varying eccentricities. In addition, over 400 tests were performed on 457 mm (18 in.) long specimens in order to calibrate a finite element analysis for longer specimens. These tests agreed well with a subset

* Corresponding address: 3122 Newmark Civil Engineering Lab, 205 North Matthews Ave, Urbana, IL 61801, USA. Tel.: +1 217 244 4178; fax: +1 217 265 8039.

E-mail addresses: dborello@illinois.edu (D.J. Borello), andrawes@illinois.edu, andrawes@uiuc.edu (B. Andrawes), jfhajjar@illinois.edu (J.F. Hajjar), olson@illinois.edu (S.M. Olson), jrh@shanwil.com (J. Hansen).

¹ Newmark Civil Engineering Lab, 205 North Matthews Ave, Urbana, IL 61801, USA.

² 400 N 34th Street, Suite 100, PO Box 300303, Seattle, WA 98103-8600, USA.

³ Formerly at Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana, IL, USA.

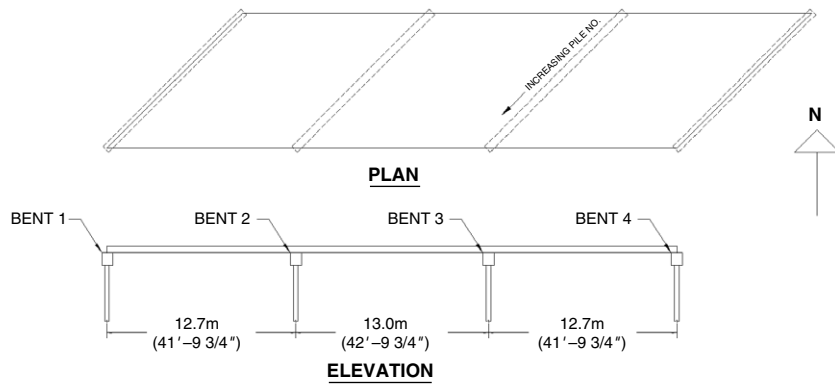


Fig. 1. Prototype bridge layout.

of 2438 mm (96 in.) long experimental tests. The short specimen compressive strength was reduced by 57% when the load eccentricity was increased from 25 mm (1 in.) to 89 mm (3.5 in.).

Zahn [10] derived interaction equations to include second-order effects coupling the axial force and bending moment, similar to the AISC (1986) [11] procedure. These equations reasonably captured experimental results and were adopted by the current National Design Specification [12].

The experimental testing of wood under combined compression and flexure has concentrated on small sawn lumber. There has been limited research on large circular specimens typical of timber piles. This work analyses the response of timber piles under combined compression–flexure loading. The effects of pile aging/deterioration and moisture content are considered in the study. The results of relatively short specimens are extended numerically using finite element analysis to explore the response of longer piles.

3. Prototype bridge

A recently collapsed rural bridge in DeKalb County, Illinois was used as a prototype structure for this work (Fig. 1). The prototype bridge deck was simply supported; therefore its supporting piles were susceptible to eccentric loads under unsymmetrical loading conditions such as the one shown in Fig. 2. The bridge was constructed in 1976 to service local agricultural traffic based on 1973 AASHTO HS-20 load criteria [13], which did not explicitly account for potential load eccentricity.

As shown in Fig. 1, the bridge consisted of three 12.8 m (42 ft) spans skewed 45° left forward and traversed a small stream (that flowed between bents 2 and 3) used for agricultural drainage. The bents and piles were numbered increasing to the east and south respectively. The deck was 432 mm (17 in.) thick precast pretensioned concrete beams supported on concrete pile caps connected by 19 mm (3/4 in.) dowels cast in the pile cap and inserted into 51 mm (2 in.) holes in the deck beam. This connection provided negligible rotational restraint between the deck beams and pile cap. The timber piles were embedded 305 mm (12 in.) in the cast-in-place pile cap, providing the ability to transfer axial force and moment. The circular timber piles each had a 254 mm (10 in.) nominal diameter and were 8.5 m (28 ft) long embedded approximately 5.2 m (17 ft) below the stream mudline.

The timber piles were made from oak, although the specific species of timber was not specified in the design documents. The original bridge plans required the timber piles to have a 214 kN (48 kip) nominal capacity. As part of the regular maintenance of the bridge, in 2000 the top six feet of two piles of bent 3 were removed due to severe deterioration and replaced by new round timber posts.

During the removal of the bridge after its recent collapse, the researchers were able to retrieve several samples of the bridge piles. These samples were used to determine the strength of the timber piles experimentally under compression and combined compression and flexure and the test results were used to calibrate accurate models for the prototype bridge piles.

4. Laboratory testing

4.1. Specimen description

Eight pile specimens from the collapsed structure were retrieved in order to investigate the influence of eccentricity on timber pile strength; their properties are summarized in Table 1. Six specimens were retrieved from three piles of bent 3 and two of the specimens were cut from bent 2. The specimens were designated as being either above or below the riverbed or from one of the two posts inserted into the piles. The specimens were each cut to a length of 914 mm (36 in.), but the cross-section dimensions varied slightly between samples due to the taper and natural irregularities in the timber piles. The cut surfaces were cut as smoothly and squarely as practical to ensure uniform loading. The specimens were tested at two moisture content conditions, air-dried and water-saturated, to evaluate the influence of moisture content on the pile mechanical properties above and below the water level. Air-dried specimens were dried for 36 days in the laboratory, while saturated specimens were submerged in water and weighed periodically until their weight stabilized, which took approximately 8 days.

The specimens were subjected to either monotonic or cyclic compression loading or monotonic combined compression and flexure, as included in Table 1, to directly evaluate the effects of timber age, location and moisture content (i.e., above or below the riverbed), and loading type. Specimens SP1 and SP2 compare the response of the posted material to the original piles. Specimens SP4 and SP6 compare pile sections above and below the riverbed, at their appropriate natural moisture content. Specimens SP3 and SP8 compare sections from above and below the riverbed at the same moisture content, under combined compression and flexure. Specimens SP1 and SP5 as well as SP6 and SP7 compare the effect of monotonic and cyclic testing for air-dried and saturated conditions, respectively. Specimens SP6 and SP8 compare the response with additional eccentricity.

4.2. Strength prediction

Prior to conducting the experimental tests, prediction of pile (and specimen) strengths was conducted per the National Design Specification (NDS) for Wood Construction Section 6, Round

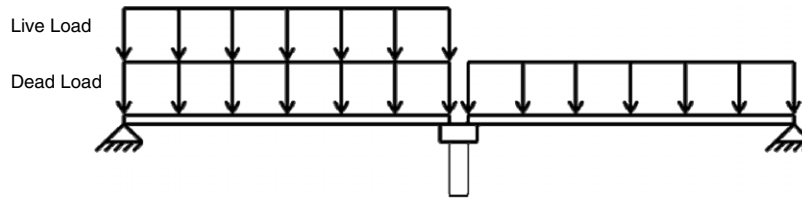


Fig. 2. Unsymmetrical load of bridge.

Table 1

Test specimen matrix.

Specimen	Bent number ^a	Pile number ^b	Elevation ^c	Moisture content	Test type	Loading plan
SP1	3	1	Post	Air-dried	Compression	Monotonic
SP2	2	4	Above	Air-dried	Compression	Monotonic
SP3	2	4	Above	Saturated	Compression–flexure	Monotonic
SP4	3	4	Above	Air-dried	Compression	Monotonic
SP5	3	1	Post	Air-dried	Compression	Cyclic
SP6	3	1	Below	Saturated	Compression	Monotonic
SP7	3	1	Below	Saturated	Compression	Cyclic
SP8	3	2	Below	Saturated	Compression–flexure	Monotonic

^a Bents are numbered increasing to the east.^b Piles are numbered increasing to the south.^c Elevation describes the original position of the specimen as either above the water line, below the water line, or a repair post.

Timber Poles and Piles [12]. Load and Resistance Factor Design (LRFD) procedures were used to determine the predicted ultimate strength without the use of a resistance factor. Reference design values of compressive strength and elastic modulus for Red Oak [12] were utilized as the basis for the calculations. For specimens tested in compression, the predicted strength is the product of the reference compression strength parallel to the grain (7.58 MPa (1100 psi)) and the LRFD conversion factor (2.4), which equals 18.3 MPa (2640 psi), neglecting any reduction due to flexural buckling since the specimens were short. The LRFD conversion factor translates the tabulated reference values from an allowable strength design to an ultimate strength.

The predicted strength of the in-situ pile is the section strength adjusted to account for flexural buckling. For the prototype bridge, the pile length above the riverbed is approximately 3.3 m (11 ft). The deck provided negligible rotational resistance to the pile cap, allowing the top of the pile cap rotate. Assuming the pile is restrained against rotation by the stiff glacial soils [approximately 1 m (3 ft) below the riverbed] and pinned at the pile cap, the effective length factor can be conservatively taken as unity. Therefore the effective unbraced length is approximately 4.3 m (14 ft). Based on the NDS column stability curve, the compressive strength reduction is 36% due to flexural buckling [12]. Therefore the predicted ultimate compressive strength, based on a 254 mm (10 in.) nominal pile diameter, of the full length pile is approximately 592 kN (133 kips), or 2.8 times the required nominal capacity of 213 kN (48 kips). Thus, it is likely that original pile design was controlled by soil capacity, not pile strength [3].

4.3. Test description

Tests were conducted per ASTM D198 [14] on a 2.7 MN (600 kip) MTS uniaxial hydraulic frame. For compression tests, a spherical head was placed below the specimen to prevent unintentional loading eccentricities. The specimens were instrumented with two longitudinal extensometers and a circumferential extensometer along with the machine's internal actuator position linear variable differential transformer (LVDT) and load cell as shown in Fig. 3. The monotonic tests were initially conducted under displacement control at a rate of 1.0 mm (0.04 in.) of cross-head displacement per minute to obtain an overall wood fiber strain rate of 0.001 mm/mm/min. After substantial post-peak softening the imposed displacement rate was increased.

If the prototype bridge was placed under unsymmetrical loading conditions (Fig. 2), the piles were expected to be subjected to combined compression and bending. Compression–flexure tests were conducted to determine the response of timber piles under such loading condition. The combined compression–flexure test specimens were instrumented similar to the compression specimens, but without a circumferential extensometer. To apply the load eccentrically, specimens were bolted to 38 mm (1 1/2 in.) thick steel plates (loading plates) on each end using five 19 × 152 mm (3/4 × 6 in.) bolts as shown in Fig. 4. The plates were loaded through 51 mm (2 in.) rollers placed 76 mm (3 in.) eccentric to the centroidal axis, thereby inducing a constant moment equal to the product of the applied load and 76 mm (3 in.) combined with the axial force.

Cyclic tests were conducted to evaluate potential pile deterioration due to repeated traffic loads. The specimens were initially loaded to the approximate dead load (including approximate adjustments for the reduced length effects), and then the load was cycled twenty times to twice the dead load at the same strain rate as the monotonic tests (1 mm/min). The peak cyclic load then was increased and the specimen further cycled. The peak load for each cycle was constant since the test was executed to achieve target loads. The tests were then switched to displacement control and cycled once for each target displacement at 0.033 Hz, approximately a 15 mm/min cross-head rate at peak load.

4.4. Test results

Table 2 summarizes the compression test results. All of the timber specimens exhibited high strength and ductility. The test-to-predicted ratio describes the experimental strength normalized by the NDS specifications strength, which yielded a mean of 1.14 and standard deviation of 0.13. The mean measured specimen strength of 1108 kN (249 kips) was 5 times larger than the required pile capacity of 214 kN (48 kips). Further experimental details are presented by Borello et al. [15].

The load–displacement response of the monotonic compression tests are shown in Fig. 5a. The initial stiffness was similar and approximately linear for all specimens, while the peak load varied slightly which was influenced by cross-sectional variation between specimens. The four specimens tested monotonically yielded an average peak stress of 19.7 MPa (2861 psi). To compute the stress, the specimen area was estimated by measuring the minimum

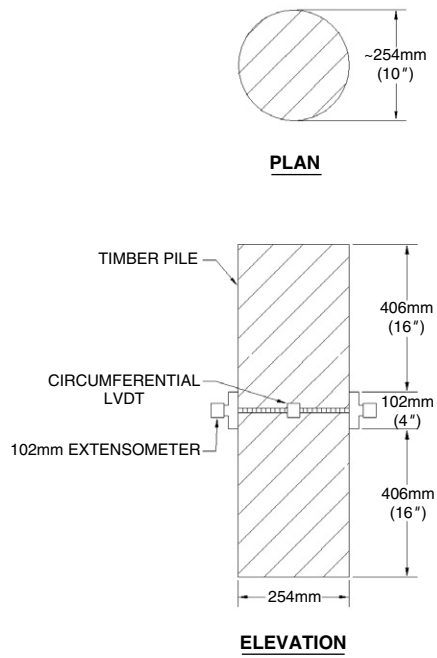


Fig. 3. Compression test setup and instrumentation.

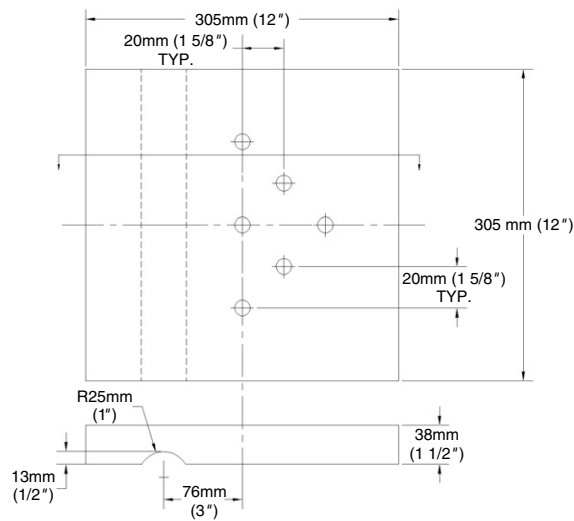


Fig. 4. Compression–flexure test setup and instrumentation.

Table 2
Compression tests results.

Specimen	Minimum diameter mm (in.)	Predicted stress MPa (psi)	Predicted strength kN (kips)	Measured ultimate stress MPa (psi)	Measured ultimate strength kN (kips)	Test/predicted ratio
SP1	275 (10.82)	18.3 (2640)	1080 (243)	17.3 (2515)	1029 (231.4)	0.95
SP2	252 (9.91)	18.3 (2640)	905 (204)	22.4 (3255)	1116 (250.9)	1.23
SP4	275 (10.84)	18.3 (2640)	1084 (244)	20.2 (2924)	1201 (270)	1.11
SP5	275 (10.84)	18.3 (2640)	1084 (244)	23.4 (3390)	1392 (313)	1.28
SP6	241 (9.47)	18.3 (2640)	827 (186)	19.0 (2752)	862 (193.8)	1.04
SP7	244 (9.61)	18.3 (2640)	852 (191)	22.1 (3206)	1034 (232.5)	1.21
			Mean	20.7 (3007)	1108 (249)	1.14
			Std. dev.	2.3 (335)	178 (40)	0.13

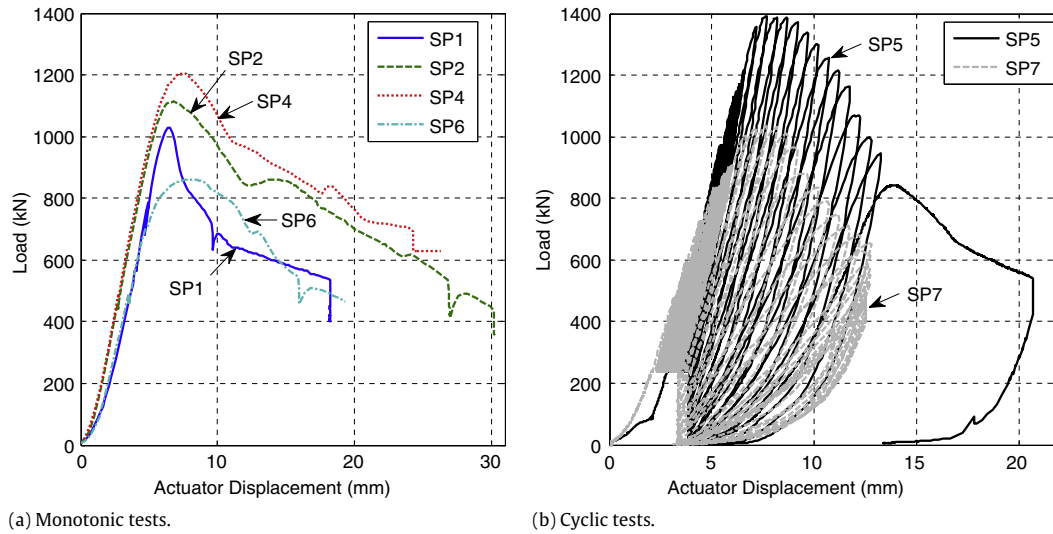


Fig. 5. Response of compression specimens.

circumference of the specimen and computing the area assuming the section was circular, which slightly overestimated the area for the non-circular specimens. The post-peak softening response was also nearly linear, exhibiting a ductile response. The slight increases in the load during this softening branch of the curve, as observed in SP2 and SP4, occurred due to an increased imposed displacement rate. Vertical drops in load represent specimen relaxation as the actuator position was held temporarily to permit specimen observation. The specimens exhibited local buckling of the fibers and occasional longitudinal splitting, shown in Fig. 6. The displacement was increased until the load decreased to 50% of the peak load. The specimens did not demonstrate catastrophic failure.

Fig. 5b presents the load–displacement response of the cyclic compression tests. The pre-peak behavior is relatively linear elastic, designated by the specimen tracing the loading path when unloaded on each cycle. Therefore no signs of cyclic degradation in strength or stiffness due to repeated loading were observed. The monotonic load–displacement curve roughly envelopes the cyclic response of a comparable specimen. When a monotonic increase in displacement was applied, the response continued to follow the monotonic backbone curve.

Table 3 presents the compression test results by type. The natural variability of wood combined with the small number of tests precludes determining conclusive trends regarding how specific variables affected timber pile capacity, but the following observations can be made. The 17% increase observed in cyclic strength (compared to monotonic tests) can be attributed to the increased imposed strain rate at peak strength, as all specimens demonstrated an increased strength when strain rate was increased during the test. Up to the fiber saturation point, timber strength is inversely related to the moisture content [16,2]. However, air-dried and saturated specimens in this test program yielded similar

results, suggesting that the tested specimens were above the fiber saturation point.

The two specimens, SP3 and SP8, which were tested in combined compression and flexure, were equipped with two extensometers placed symmetrically about the axis of bending. As expected, one side of the specimen experienced net tension while the other experienced compression. The applied load versus the cross-head displacement response of the two specimens is presented in Fig. 7. The response was similar to the monotonic compression behavior, exhibiting approximately linear pre-peak and post-peak behavior.

Table 4 summarizes the compression–flexure test results. In the table, the predicted strength is calculated by solving the NDS [12] interaction equation [Eq. (1)] with the 76 mm (3 in.) eccentricity for the stress due to the axial load.

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_c(6e/d)[1 + 0.234(f_c/F_{cE})]}{F'_b[1 - (f_c/F_{cE})]} \leq 1.0 \quad (1)$$

where:

f_c = stress due to axial load

F'_c = predicted compressive strength

F_{cE} = compression critical buckling design value = $\frac{0.822E'_{\min}}{(l/d)^2}$

$E'_{\min}$ = adjusted modulus of elasticity for stability
(calculated per NDS)

l = unbraced length

d = equivalent square column width = $0.88d_{\text{round}}$

e = eccentricity of load.

Table 3
Compression tests results by type.

Test type	Mean ultimate stress MPa (psi)	Mean ultimate strength kN (kips)	Mean test/predicted ratio
Monotonic	19.7 (2861)	1054 (237)	1.08
Cyclic	22.7 (3298)	1214 (273)	1.25
Air-dried, monotonic	20.0 (2898)	1117 (251)	1.10
Saturated, monotonic	19.0 (2752)	863 (194)	1.04
Untreated, monotonic	18.2 (2634)	948 (213)	1.00
Posts, monotonic	21.3 (3089)	1157 (260)	1.17



Fig. 6. Specimen after peak strength.

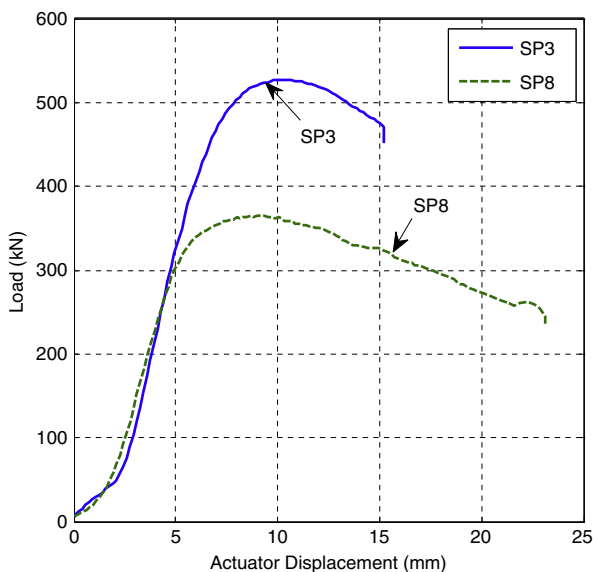


Fig. 7. Average stress vs. average strain response of specimens tested in compression-flexure.

Table 4
Compression-flexure tests results.

Specimen	Predicted strength kN (kips)	Ultimate strength kN (kips)	Test/predicted ratio
SP3	703 (158)	533 (119.8)	0.76
SP8	494 (111)	366 (82.3)	0.74
Mean		449 (101)	0.75
Std. dev.		120 (27)	0.01

The mean strength of the two specimens was 449 kN (101 kips), showing a 60% reduction from the concentrically loaded specimens. This result illustrated that the timber pile strength is sensitive to even modest load eccentricity. The mean test-to-predicted ratio of 0.75 may be attributed to a reduced specimen cross-section due to damage and longitudinal splitting, which slightly increased the calculated cross-sectional area and moment of inertia used to compute the predicted strength. However, this indicates that the NDS interaction equation may underestimate the reduction due to load eccentricity.

5. Numerical simulation

A numerical model was created for each of the experimental tests, and a single in-situ pile of the prototype bridge using the nonlinear finite element program OpenSees [17]. The analysis included geometric and material nonlinearity. The analyses utilized displacement-based nonlinear beam-column elements, with 5 integration points along their length. The elements comprised circular fiber sections with 5 divisions radially and 50 divisions circumferentially as shown in Fig. 8a. The constitutive behavior of each fiber was defined using a uniaxial material. A co-rotational geometric transformation was utilized to fully capture geometric nonlinearity due to moderate displacements and rotations, allowing the model to capture flexural buckling.

5.1. Models of laboratory tests

To calibrate the numerical model that will be used in the full-scale pile analysis, two numerical models were developed for the tested specimens under compression loading (Fig. 8b) and combined compression and flexure loading (Fig. 8c). The 914 mm (36 in.) specimens were represented by six 152 mm (6 in.) elements. The spherical head of the compression tests was modeled as a pinned boundary condition while the surface bearing was represented as fixed as shown in Fig. 8b. The boundaries for the combined compression-flexure tests were modeled using eccentric pinned supports as shown in Fig. 8c. Two uniaxial materials were calibrated from the experimental response of the timber specimens, representing the pile above and below the riverbed, assumed to have air-dried and water-saturated moisture contents, respectively. The compression branch was modeled using specimens SP4 and SP6 responses for above and below the riverbed, respectively, as nonlinear up to a peak stress. The post-peak response was modeled as linear softening followed by perfectly plastic behavior due to residual stresses at half of the peak

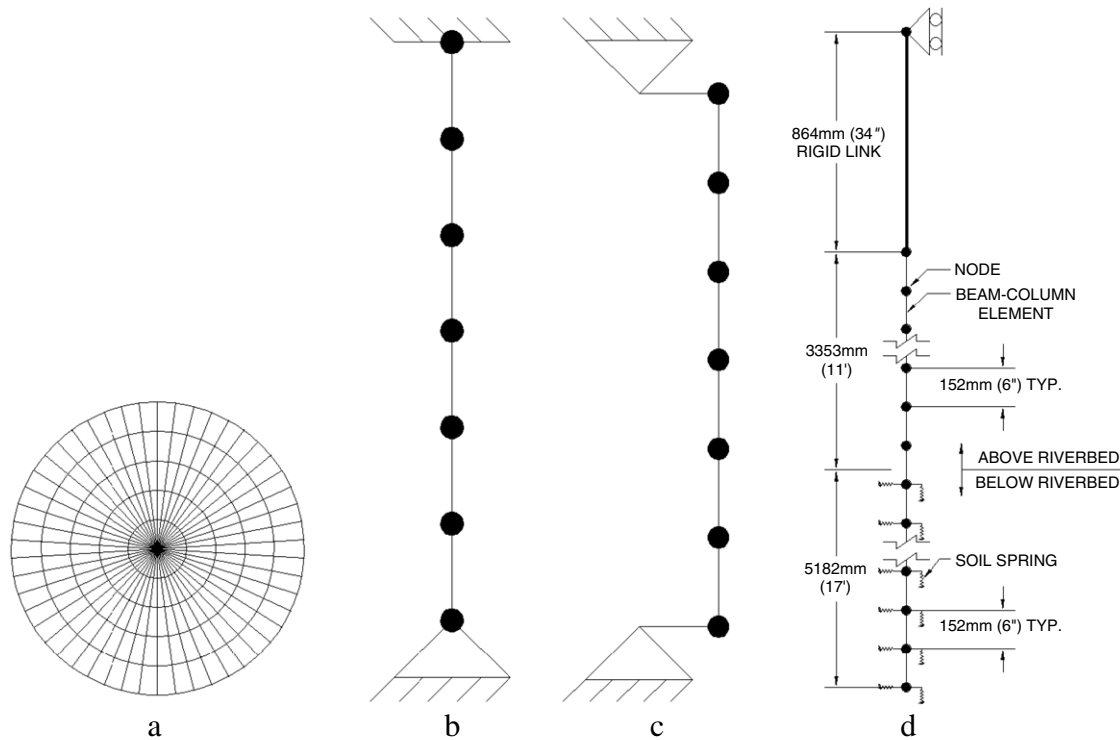


Fig. 8. Numerical models: (a) Element cross-section (b) Compression experimental specimens (c) Compression–flexure experimental specimens (d) Single pile.

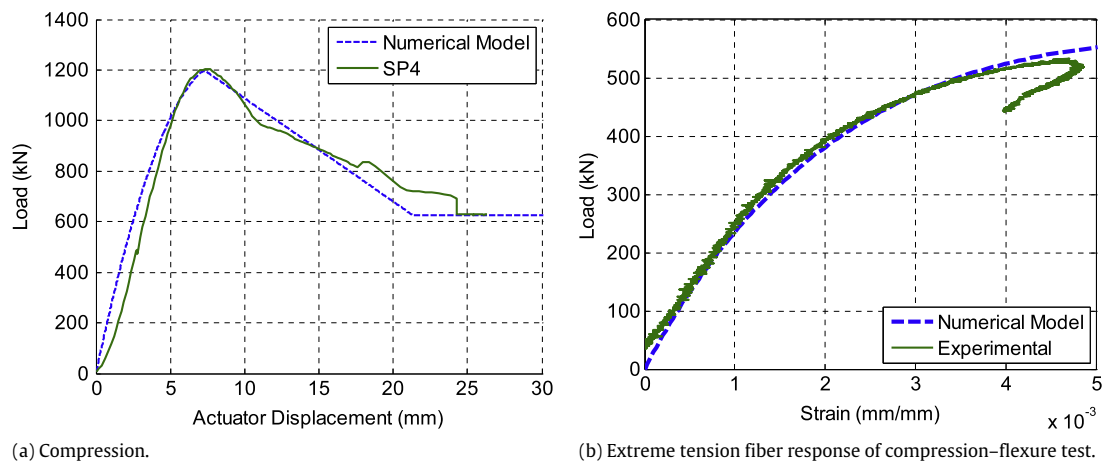


Fig. 9. Experimental vs. computational response of experimental specimens.

stress. Due to the similarities between the compressive behaviors of concrete and tested timber, the compression branch of the uniaxial material model was based on the OpenSees Concrete02 material model [17]. A comparison between the experimental and numerical behavior of specimen SP4 is presented in Fig. 9a. On the other hand, tensile response was calibrated using the combined compression–flexure tests of specimens SP3 and SP8 for above and below the riverbed, respectively. Stress–strain response was modeled as linear elastic up to fracture based on the response of wood reported in the literature [16]. Fig. 9b illustrates the agreement between experimental and numerical strain in the extreme tensile fiber of a compression–flexure test recorded by the extensometer and predicted by the numerical model.

5.2. Prototype single pile model

For the case of a statically determinate superstructure, common for simply supported spans, the superstructure provides negligible

resistance to collapse of the foundation. Therefore, it is deemed acceptable to model the foundation independently of the superstructure. A nonlinear numerical model including soil response from the prototype structure was developed for a single pile. This model was utilized to predict the ultimate pile strength under concentric and eccentric loading conditions. A base model was developed that was deemed most representative of the likely properties of the timber piles such as diameter, load eccentricity, and out-of-plumbness ratio. However, due to uncertainties associated with selecting parameter values for the base pile model, a separate parametric study was conducted to assess the sensitivity of the developed model to these parameters.

The 2-dimensional (2-D) pile model was divided into two regions, 3.3 m (11 ft) above the riverbed, and 5.2 m (17 ft) below the riverbed based on the calibrated uniaxial material models discussed above. The pile was divided into 15 mm (6 in.) elements as shown in Fig. 8d. The top of the pile was connected to an 864 mm (34 in.) rigid link to represent the concrete pile cap. Since

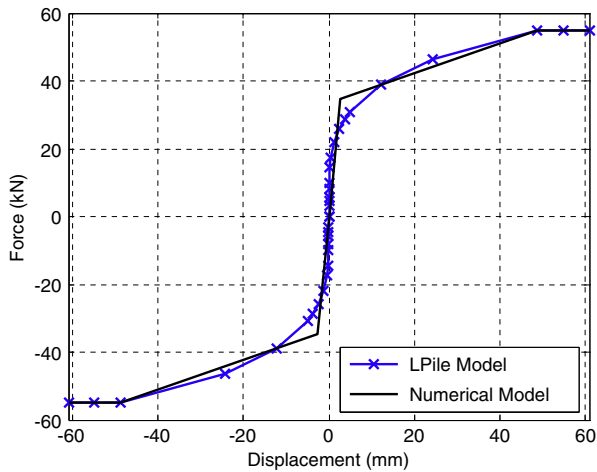


Fig. 10. Typical geotechnical and numerical model load–deformation response of lateral soil spring [2.4 m (93 in.) below the river bed].

the timber piles were slightly tapered as demonstrated in the slightly nonprismatic samples that were obtained, the model was tapered from 234 mm (9.23 in.) diameter at the pile tip to 252 mm (9.91 in.) diameter at the pile cap. The deck and cross-bracing between the piles were assumed to provide adequate stiffness to prevent longitudinal and transverse translation of the top of the pile group, respectively. However, the deck-to-pile cap connection was insufficient to restrain the pile cap against rotation. Therefore, the top node of the rigid link was constrained against horizontal translation but allowed to rotate (Fig. 8d).

Lateral (transverse) nonlinear soil springs were placed at each node [i.e., every 152 mm (6 in.)] to represent the resistance of the soil against pile buckling. The lateral springs only resisted lateral translation and were not influenced by vertical displacement. Based on soil stratigraphy and properties determined from a geotechnical site investigation performed by the Illinois Department of Transportation and the authors [15], widely-used nonlinear soil resistance–deflection relations were assigned to each spring. These soil resistance–deflection curves are commonly referred to as p - y curves, and are a function of soil type, soil density, soil strength, soil stiffness, and effective confining stress [19, 20 among others]. Specifically, the lateral springs from 0 to 915 mm (0 to 36 in.) below the riverbed were modeled using the “API RP2A” [18,19] sand model [with an effective unit weight of 9.4 kN/m³ (60 lb/ft³) and an effective friction angle of 30°] to represent the loose, sandy alluvial soils encountered in the riverbed. The lateral springs from 915 mm to 5.2 m (36 in. to 17 ft) below the riverbed were modeled using the “stiff clay with no free water” model [20,21] [with an effective unit weight of 12.3 kN/m³ (78 lb/ft³) and an unconfined compressive strength of 430 kPa (9 ksf)] to represent the very stiff to hard glacial tills underlying the alluvium. All of the nonlinear geotechnical soil springs (i.e., p - y curves) were approximated with a tri-linear constitutive formulation as shown in Fig. 10. As only one lateral spring was attached to each node, rather than a spring on both sides, the lateral springs were modeled to provide symmetric behavior in compression and tension.

Elastic–perfectly plastic vertical springs (i.e., t - z curves) were used to model skin friction and end bearing resistance of the soil, and were placed at every node below the riverbed. The vertical springs were modeled using the method of Olson [22] to estimate a maximum side resistance in the loose, sandy alluvium [with an interface friction angle of 20°], the API [19] method to estimate a maximum side resistance in the tills, and the Reese and O’Neill [23] method to estimate the maximum end bearing in the tills [both

Table 5

Numerical total and live load strength of single pile model.

	Concentric loading (Case 1) kN (kips)	Eccentric loading (Case 2) ^a kN (kips)
Pile capacity	389 (87)	201 (45)
Dead load	142 (32)	142 (32)
Live load capacity	247 (55)	58 (13)

^a 197 mm (7.75 in.) live load eccentricity.

using an unconfined compressive strength of 430 kPa (9 ksf) in the till]. Displacements of 2.5 mm (0.1 in.) and 3.0 mm (0.12 in.) were required to reach maximum side resistances and maximum end bearing [i.e., plastic response], respectively.

5.3. Loading pattern

The tributary dead load for the prototype bridge was calculated to be 1152 kN (259 kips) per bent, or 144 kN (32 kips) per pile. Since the dead load was symmetric about the pile cap, it was applied concentrically to the model pile in a static analysis over 32 load steps. Live load was applied subsequently.

When considering the live load supported by the bridge, there are two possible loading cases: (1) symmetrical, when the two spans attached to a bent are loaded equally; and (2) unsymmetrical, when the spans are unequally loaded. In the case of unsymmetrical loading, when the spans are simply supported, the live load will be eccentric to the supporting piles (Fig. 2). For the prototype structure, assuming that the deck beams were bearing uniformly on the pile cap, they would apply the live load at an eccentricity of 197 mm (7.75 in.). Therefore, the model was subjected to concentric compression for the symmetrical loading case (Case 1) and eccentric compression for the unsymmetrical case (Case 2). For Case 2, the live load was applied to the model as an axial load and a moment equal to the product of the axial load and the eccentricity. Live loads (axial and moment) were applied monotonically, up to failure, in a static analysis under displacement control to capture material softening behavior.

5.4. Single pile analysis results

Table 5 and Fig. 11 present the numerical results from the pile model under concentric (Case 1) and eccentric (Case 2) loads. Under increasing loading, the pile begins to soften, and it experiences a peak strength at which failure of the pile would be predicted to occur. The first row of Table 5 shows the peak load (capacity) achieved in the pile after both dead and live loads were applied. The dead load due to the superstructure is 142 kN (32 kips) per pile, so the balance of the load is due to the live load, shown in the third row. The results shown indicate that a single pile had a 307 kN (55 kips) and 58 kN (13 kips) live load capacity (i.e., in excess of dead load) when loaded concentrically and eccentrically, respectively. The capacity of the pile under eccentric load [i.e. 201 kN (45 kips)] is less than the 214 kN (48 kips) pile nominal capacity shown on the bridge design plans. Therefore, the design of this prototype bridge is deemed unconservative under eccentric loads, and thus Case 2 should govern the foundation design. These results clearly illustrate that flexure in the piles is a key component in the behavior of bridge timber pile behavior. The deformed shape of the pile shown in Fig. 12a confirms that the soil provided adequate resistance against buckling. The moment diagram shown in Fig. 12b demonstrates that the maximum moment occurs 610 mm (24 in.) below the pile cap. As anticipated, the computed moment below the riverbed is significantly smaller than above the riverbed.

Table 6
Sensitivity analysis parameters and results.

Parameter	Reference value (Case 2) ^a	Sensitivity analysis value	Pile strength kN (kips)	Strength reduction (%)
Diameter	Tapered from 234 mm (9.23 in.) at pile tip to 252 mm (9.91 in.) at pile top	Constant 234 mm (9.23 in.)	195.3 (43.9)	2.7
Live load eccentricity	197 mm (7.75 in.)	259 mm (10.2 in.) ^b	190.4 (42.8)	5.1
Out-of-plumb	L/500	L/48	200.2 (45.0)	0.4

^a Reference Case 2 pile strength is 201 kN (45.1 kips).

^b Represents triangular bearing distribution of deck on to pile cap.

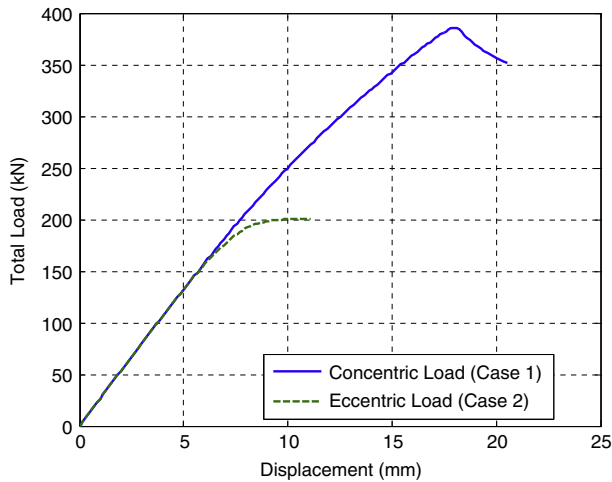


Fig. 11. Numerical pile load versus displacement results.

5.5. Sensitivity analysis

Sensitivity of the results was evaluated by varying several key parameters, as illustrated in Table 6 for Case 2. The results of the sensitivity analysis for Case 1 are not discussed, because they did not influence the critical pile capacity. The parameters examined included: (1) pile diameter; (2) eccentricity; and (3) out-of-plumbness ratio. The pile diameter varied from a tapered diameter [234 mm (9.23 in.) at the tip to 252 mm (9.91 in.) at the pile cap] to a constant value of 234 mm (9.23 in.), which corresponds to the minimum diameter observed in the retrieved samples. This analysis yielded a 2.7% reduction in pile strength. However, timber piles are typically tapered, justifying the tapering in the base model [3].

The second parameter investigated was live load eccentricity. The prototype bridge drawings illustrated that the deck bearing length was 368 mm (14.5 in.) with an offset of 13 mm (0.5 in.) from the pile cap center. For the base case, the deck was assumed

to apply a uniform distribution to the pile cap, therefore the resultant applied force was located 197 mm (7.75 in.) from the pile group centerline. However, if the deck was assumed to apply a triangular distribution to the pile cap, the distance from the pile group centerline to the centroid of the triangular force distribution is 259 mm (10.2 in.). The increased eccentricity resulted in a 5.1% drop in pile strength. However, bearing is typically considered to be uniform and there was no evidence to the contrary observed for the prototype bridge.

The final parameter considered in this study was the out-of-plumbness ratio of the driven piles, which can be as large as L/48. In this study, the extreme out-of-plumbness ratio would result in an offset of 178 mm (7 in.) from the pile tip to top. Although the base model was negligibly out-of-plumb (L/480 to ensure flexural buckling), the analysis proved insensitive to initial imperfections, showing only a 0.2% strength reduction due to out-of-plumbness of L/48.

As illustrated by the sensitivity analysis results in Table 6, the base model is not susceptible to large fluctuations due to variations in the pile parameters. Therefore the results of the base model were deemed reasonable.

6. Conclusions

This paper summarizes the experimental and numerical work conducted to explore the effect of loading eccentricity on the strength of timber pile bridge foundations. Timber piles are not required to be designed or rated for combined compression and flexure loading by current AASHTO design provisions although eccentricities are typically present for unsymmetrical loading of simply supported spans. A recently collapsed bridge was used as a prototype in this study and also to provide the experimental specimens needed for testing. The bridge consisted of three 42-foot spans simply supported by concrete pile caps founded on timber piles.

Six pile samples were retrieved for experimental testing. Eight 914 mm (36 in.) long specimens were tested under pure

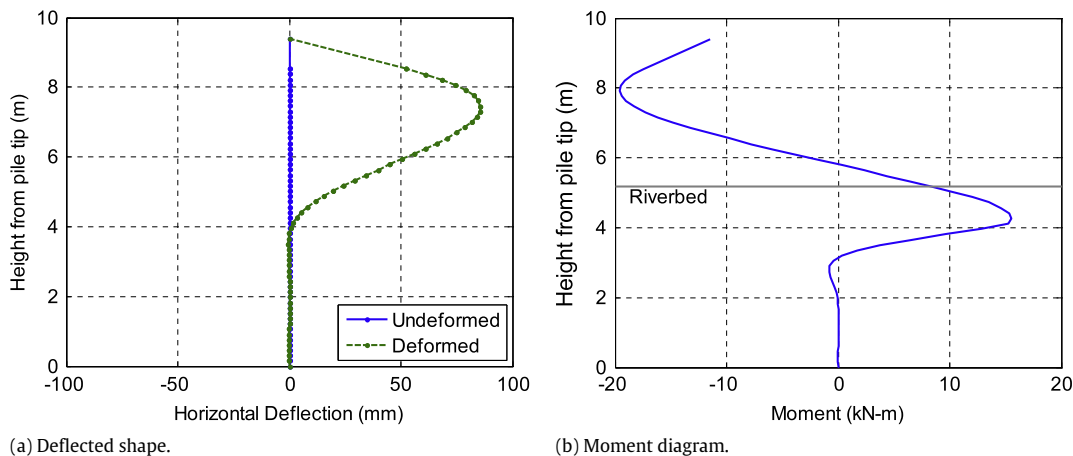


Fig. 12. Deflected shape and moment diagram of a single pile under eccentric load.

compression and combined compression and flexure loading. The six specimens that were tested under pure compression exhibited a mean capacity of 1108 kN (249 kips). The two specimens tested in compression with a 76 mm (3 in.) eccentricity showed a 60% reduction in strength compared to the concentrically loaded specimens. This result clearly illustrates the sensitivity of timber pile capacity to eccentric loading.

The experimental results were used to calibrate a numerical 2-D finite element model that was used to study the behavior of a full-scale single pile embedded in the soil. The single pile model consisted of a series of 152 mm (6 in.) beam–column elements and was supported by nonlinear lateral and vertical soil springs based on the soil conditions. The timber pile material constitutive relationship was calibrated using the experimental results. The analysis included geometric and material nonlinearity. The pile was loaded with a constant dead load of 144 kN (32 kips) and an incrementally increasing live load until failure.

The numerical model indicated that a single pile had a 307 kN (55 kip) and 58 kN (13 kip) live load capacity (i.e., in excess of dead load) when loaded concentrically and eccentrically, respectively. The total eccentric load capacity of the pile (dead load plus live load) was 201 kN (45 kips), which is less than the maximum permitted load of 214 kN (48 kips) per the design drawings. Therefore, neglecting flexure in pile bent design for simply supported spans can result in unconservative designs. It is important to note that even though the design load is one-fifth of the compressive strength obtained from experimental tests [1108 kN (249 kips)], the design of the bridge was still unconservative under eccentric loads, which illustrates that using an arbitrary factor of safety as high as 5 could still result in unsatisfactory design in some cases. This is mainly since the reduction in the load-carrying capacity due to the application of eccentric loading highly depends on the material and geometric parameters that could vary for each case. Therefore, based on this experimental and numerical work, it is recommended that eccentrically applied loads should be considered when determining the capacity of bridges supported on timber piles, especially when the superstructure is simply supported.

Acknowledgements

This work was supported by the Illinois Center for Transportation (ICT); the Illinois Department of Transportation (IDOT); the US DOT, the Federal Highway Administration (FHWA); and the University of Illinois at Urbana-Champaign. The authors thank the members of the project Technical Review Panel from IDOT, FHWA, ICT, as well as personnel from IDOT and DeKalb County Highway Department for their assistance with the research.

References

- [1] Timber Piling Council. Timber pile design and construction manual. Vancouver (WA): American Wood Preservers Institute; 2002.
- [2] Forest Products Society. Wood handbook – wood as an engineering material. Madison (WI): Forest Products Society; 1999.
- [3] ASTM International (ASTM). D25 standard specification for round timber piles. West Conshohocken (PA): American Society of Testing and Materials; 1999.
- [4] Tomlinson MJ. Pile design and construction practice. London (UK): Viewpoint Publications; 1977.
- [5] Peck RB, Hanson WE, Thornburn TH. Foundation engineering. New York (NY): John Wiley & Sons Inc; 1974.
- [6] American Association of State Highway Officials (AASHTO). LRFD bridge design specification. Washington (DC): American Association of State Highway Officials; 2007.
- [7] Newlin JA, Trayer GW. Stresses in wood members subjected to combined column and beam action. Madison (WI): Forest Products Laboratory; 1956.
- [8] Wood LW. Formulas for columns with side loads and eccentricity. Madison (WI): Forest Products Laboratory; 1950.
- [9] Zahn JJ. Strength of lumber under combined bending and compression. Madison (WI): Forest Products Laboratory; 1982.
- [10] Zahn JJ. Combined-load stability criterion for wood beam–columns. J Struct Eng, ASCE 1988;114(11):2612–28.
- [11] American Institute of Steel Construction (AISC). Load and resistance factor design specification for structural steel buildings. Chicago (IL): American Institute of Steel Construction, Inc.; 1986.
- [12] American Forest and Paper Association (AFPA). National design specification for wood construction. Washington (DC): American Forest and Paper Association; 2005.
- [13] American Association of State Highway Officials (AASHTO). Standard specifications for highway bridges eleventh edition. Washington (DC): American Association of State Highway Officials; 1973.
- [14] ASTM International (ASTM). D198 standard testing methods of static tests of lumber in structural sizes. West Conshohocken (PA): American Society of Testing and Materials; 2005.
- [15] Borello DJ, Andrawes B, Hajjar JF, Olson SM, Hansen J, Buenker J. Forensic collapse investigation of a concrete bridge with timber piers. Springfield (IL): Illinois Center for Transportation; 2009. <http://ict.illinois.edu/Publications/report%20files/FHWA-ICT-09-042.pdf>.
- [16] Gurfinkel G. Wood engineering. New Orleans (LA): Southern Forest Products Association; 1973.
- [17] OpenSees. Open system for earthquake engineering simulation. Berkeley (CA): University of California; 2009.
- [18] Reese LC, Wang ST, Isenhowe WM, Arrellaga JA. Computer program LPILE plus, version 5.0. Technical manual. A program for the analysis of piles and drilled shafts under lateral loads. Austin (TX): Ensoft, Inc.; 2004.
- [19] American Petroleum Institute (API). Recommended practice for planning, designing, and constructing fixed offshore platforms. API Recommended Practice 2A (RP2A). Seventeenth ed. April 1987.
- [20] Welch RC, Reese LC. Laterally loaded behavior of drilled shafts. Research Report no. 3-5-65-89. Center for Highway Research. University of Texas at Austin. May 1972.
- [21] Reese LC, Welch RC. Lateral loading of deep foundations in stiff clay. J Geotech Eng Div Amer Soc Civil Eng 1975;101(GT7):633–49.
- [22] Olson RE. Axial load capacity of steel pipe piles in sand. In: Proceedings of the offshore technology conference. 1990.
- [23] O'Neill MW, Reese LC. Drilled shafts: construction procedures and design methods. Report no. FHWA-IF-99-025. Washington (DC): Federal Highway Administration; 1999.