Application of Precast, Prestressed Concrete Piles in Integral Abutment Bridges

Mounir R. Kamel, Joseph V. Benak, and Maher K. Tadros, *University of Nebraska-Lincoln*
Mostafa Jamshidi, *Nebraska Department of Roads*

In jointless integral abutment bridge superstructures, thermally induced movements must be absorbed by the abutments, which in most cases are supported on piles. Most states in the United States use steel piles in their integral abutment bridges. Research was undertaken to compare the flexibility of steel and concrete piles to determine whether concrete piles may be used in integral abutment bridges and, if not, to modify the pile abutment joint detail currently used with steel piles for possible use with concrete piles. Load-deflection tests on one steel and two concrete piles were conducted to evaluate and compare their stiffnesses. The computer program LPILE was used to analyze both concrete and steel piles in various types of soil. The results of the analysis and tests showed that concrete piles have limited flexibility for lateral loads with current pile abutment details; therefore, they can be used only in short span integral abutment bridges. For concrete piles, a new pile abutment joint was investigated. The joint consists of a neoprene bearing pad with a Teflon layer. It allows for controlled movement or rotation of the pile relative to the abutment, or both. Laboratory tests were conducted to study the behavior of the proposed joint under axial and lateral loads. The test results showed that the proposed joint would allow the use of concrete piles in integral abutment bridges of lengths comparable to those with steel piles.

Historically, a system of expansion joints, roller supports, and other structural releases was provided in long bridges to permit thermal expansion and contraction (Figure 1*a*). Providing expansion joints in a bridge, however, leads to a substantial increase in initial cost. In addition, expansion joints are sources of deterioration and frequently do not operate as intended, thus resulting in high maintenance costs. Integral abutment bridges provide an attractive design alternative (Figure 1*b*). These are defined as bridges with no movement joints at the abutments. The supporting foundation, in most cases piles, therefore has to be flexible enough to accommodate all lateral movements that develop from thermal expansion and contraction of the bridge superstructure. The maximum thermal expansion that can be allowed by the piles without significantly decreasing their vertical load capacity or integrity is of primary importance.

Currently, most states, including Nebraska, use steel H-piles in integral abutment bridges. When used as friction piles, steel H-piles must be driven deeper than concrete piles to attain their required vertical load capacity. This is because of higher concrete/soil friction than steel/soil friction and because a concrete pile has a larger cross-sectional area. A smaller number of shorter concrete piles could provide the same amount of vertical

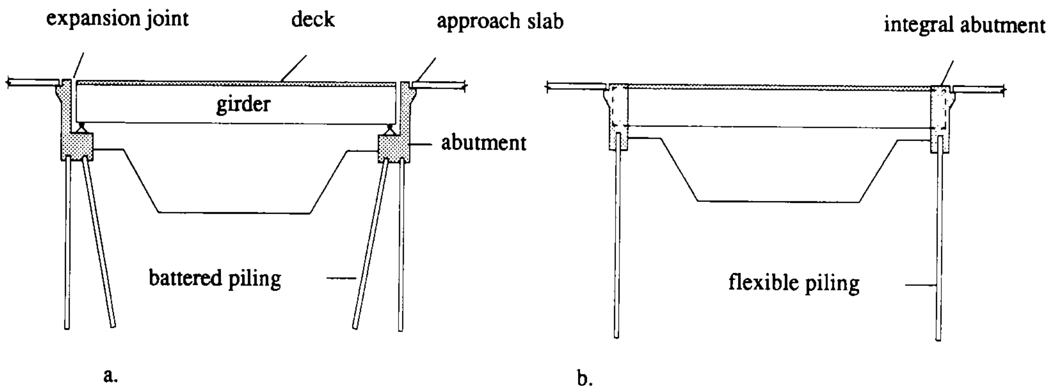


FIGURE 1 Types of bridge abutments: (*a*) bridge with expansion joints; (*b*) jointless bridge with integral abutments.

capacity as a given number of steel piles. This is true when pile loads are governed by soil bearing capacities rather than pile material properties, which is likely to happen in most practical cases.

Research Objectives

The main objectives of this research were (*a*) to evaluate the current design practices of agencies using integral abutments supported on concrete piles, especially precast concrete piles; (*b*) to determine whether precast concrete piles are feasible for use in integral bridges and if they are not; (*c*) to modify the current details that are used by various bridge designers for possible use in Nebraska.

Current Practice

A survey was conducted among highway agencies in the United States to identify those agencies that use precast, prestressed concrete piles in integral abutment bridges. The survey showed that steel, timber, concrete cast in drilled holes, concrete cast in thin steel shells, and precast concrete piles are all used in integral abutment bridges. However, most of the states prefer to use steel H-piles (Figure 2). States that use integral abutments rely on their own experience, empirical formulas, and simplified design assumptions to place span limits, instead of depending on theoretical calculations.

Many states use predrilled oversized holes filled with granular soil. It is assumed that pile stresses are relieved and allowable lengths of integral abutment bridges are increased accordingly. Various depths of these holes are required by different states. An additional feature of predrilled oversized holes is reduction of downdrag forces when compressible soil is present or minimization of the effects of elastic shortening when prestressed concrete superstructures are used, or both.

In most states where steel H-piles are used, the effect of thermal movement of the superstructure on the piles is not considered as long as bridges are designed according to their span limits and details. From a review of the literature, it was found that there is no reported specific research conducted on prestressed concrete piles in integral abutment bridges. The review included research on piles in integral abutment bridges (*1–3*), laterally loaded piles (*4–6*), and seismic design and ductility of prestressed concrete piles (*7–9*).

Laboratory Testing

Laboratory tests were conducted on three pile to pile cap specimens. The specimens were tested as cantilevers to obtain the load-deflection relationship. The objectives of the tests were to compare the stiffness of concrete piles with the stiffness of steel piles and to evaluate the stiffness *S* (equivalent *EI*) versus bending moment for concrete piles. In addition, it was necessary to study the behavior of the pile to pile cap joint in terms of whether the current joint detail provides full or partial fixity for the pile head and whether there is any relative rotation, movement, or slippage occurring at the pile to pile cap interface.

Three pile specimens were tested: one steel H-pile and two prestressed concrete piles. The steel H-pile (Specimen 1) was A36 steel 10 in. × 42 lb/ft (25.44 mm × 0.61 kN/m). The two concrete piles were 12 in. (305 mm) square, one with 9-in. (228 mm) pitch spiral reinforcement (Specimen 2) and the other with 3-in. (76-mm) pitch spiral reinforcement (Specimen 3). Pile Unit 3 represents the standard detail used in Nebraska for 12-in. (305-mm) concrete piles. The concrete pile cap dimensions and details were prepared with the same

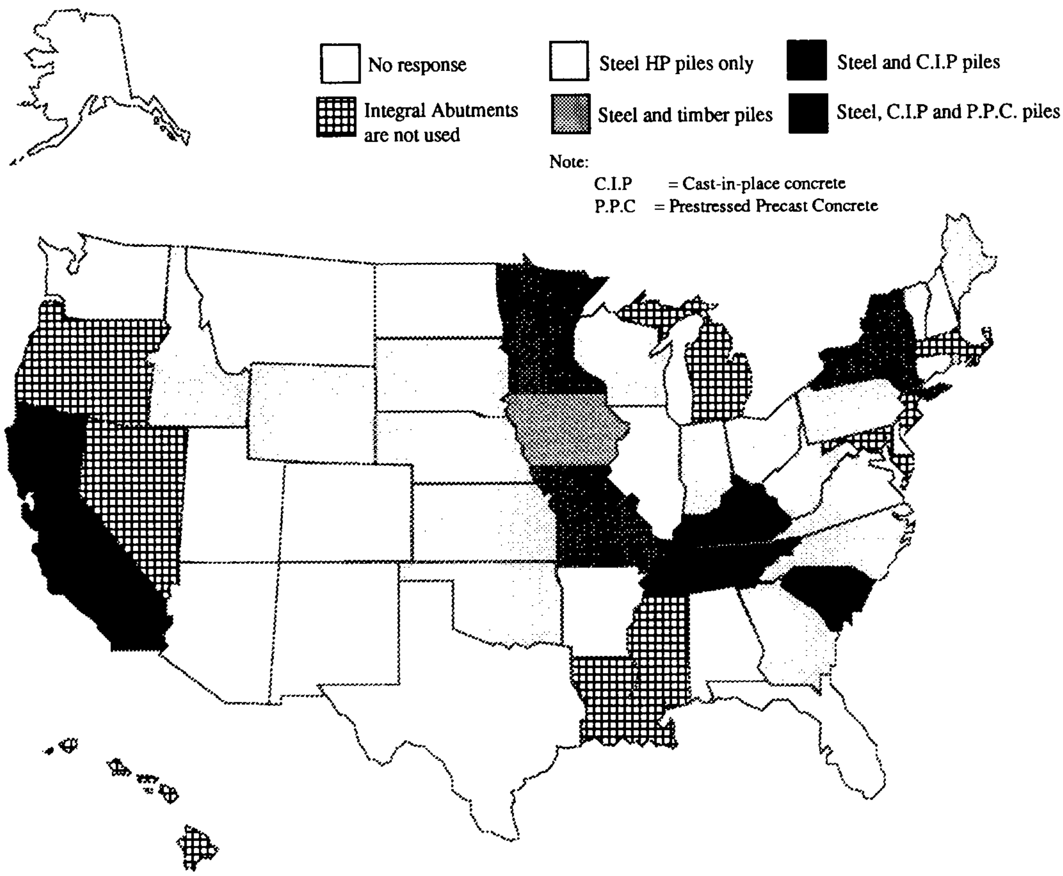


FIGURE 2 Types of piles in integral abutment bridges in various states.

standards as those currently used by the Nebraska Department of Roads so as to simulate the bridge abutment. The embedded length of both concrete and steel piles was 24 in. (610 mm). Test results are shown in Figure 3.

Stiffness of Prestressed Concrete Piles

To predict the behavior of the concrete pile at different loading stages, a nonlinear analysis was used to calculate the moment versus stiffness of the piles. The stiffness of a cracked section varies along the pile length according to the magnitude of bending moment the section is subjected to and, hence, varies according to the stage of loading. The basis of the nonlinear solution is to calculate the proper depth of compression zone for a section at a given concrete strain value. Once both the depth of compression zone and the strain distribution are obtained, all forces at the section can be calculated. These are the forces in the concrete and in the top and bottom strands as well as an applied axial load, if it exists. In these analyses, the axial applied load is 0. With a trial-and-error procedure, the proper depth of the compression zone can be obtained, and all strains and stresses at the section are known. The stiffness of the section is defined by the following equation:

$$\text{Stiffness } S\text{, equivalent } EI = \frac{M}{\theta} \qquad (1)$$

where θ is the curvature of the section, which is equal to the slope of the strain diagram and M is the moment at the section. Figure 4 shows the assumed stress and strain distribution diagrams of a pile section in a cracked stage. The nonlinear concrete stress-strain relationship is represented by Equation 2. This analytical approximation of concrete stress-strain relationship was given by Hognested (*10, 11*), as a result of his experimental study on the behavior of concrete members under combined bending and axial loads.

$$f_c = f'_c \left[2\left(\frac{\epsilon}{\epsilon_o}\right) - \left(\frac{\epsilon}{\epsilon_o}\right)^2 \right] \qquad (2)$$

The integration of Equation 2 over the depth of compression zone, Figure 4, gives the total of compression

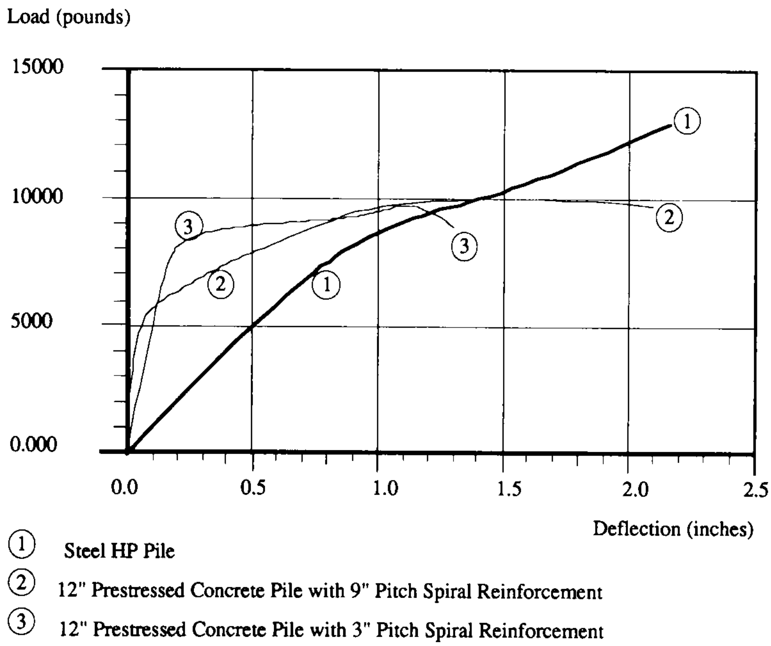


FIGURE 3 Combined load deflection test results for steel and concrete piles.

force F_c in the concrete block as follows:

$$F_c = t\int_{x=0}^{x=c} f_c d_x \tag{3}$$

substituting ϵ by (θx) in Equation 2, where $\theta = \epsilon_c/c$, and integrating gives the total compression force F_c:

$$F_c = tf'_c\left[\left(\frac{\theta}{\epsilon_o}c^2 - \frac{1}{3}\left(\frac{\theta}{\epsilon_o}\right)^2 c^3\right]\right. \tag{4}$$

The moment of F_c about the neutral axis can be given by the following equation:

$$M_{cNA} = t\int_{x=0}^{x=c} f_c \times d_x \tag{5}$$

Using the same substitutions as in Equation 4, the moment of the concrete force about the neutral axis is given by the following equation:

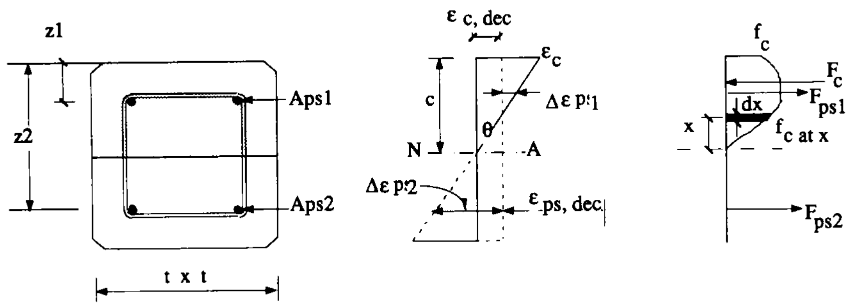
$$M_{cNA} = tf'_c\left[\frac{2}{3}\left(\frac{\theta}{\epsilon_o}\right)c^3 - \frac{1}{4}\left(\frac{\theta}{\epsilon_o}\right)^2 c^4\right] \tag{6}$$

The stress in the strands f_{ps} can be calculated from their strain values using the following equation:

$$f_{ps} = \epsilon_{ps}\left[A + \frac{B}{\{1 + (C\epsilon_{ps})^D\}^{1/D}}\right] < f_{pu} \tag{7}$$

where

ϵ_{ps} = the strain in prestressing steel and
f_{pu} = ultimate stress of prestressing steel.



a. Cracked section b. Strain distribution c. Stress distribution

FIGURE 4 Properties of prestressed cracked pile section.

A, *B*, *C*, and *D* are constants presented with the details of the equation by Devalapura and Tadros (*12*). After all losses, the strain in the strands, $\epsilon_{ps,dec}$, the decompression strain caused by the effective prestressing, is calculated from the following equation:

$$\epsilon_{ps,dec} = \frac{P_{se}}{A_{ps}E_{ps}} \tag{8}$$

where

P_{se} = effective prestressing force,
A_{ps} = area of the prestressing steel, and
E_{ps} = modulus of elasticity of prestressing steel.

The initial strain in the concrete, ϵ_{ci}, before applying any bending moment, is calculated from the following equation:

$$\epsilon_{ci} = \epsilon_{c,dec} = \frac{P_{se}}{A_c E_c} \tag{9}$$

When the section is subjected to applied moment causing compression in the concrete top fibers and tension in the bottom fibers, the top strands will be subjected to a certain compression strain, $\Delta\epsilon_{ps1}$, and the bottom strand will be subjected to a tension strain, $\Delta\epsilon_{ps2}$. Flexural moment decreases the tension strain in the top strands and increases the strain in the bottom strands. The final strain in the strands is calculated by the summation of $\epsilon_{ps,dec}$ and $\Delta\epsilon_{ps}$. A spreadsheet program was used for the calculations. Figure 5 shows the calculated moment versus stiffness relationship.

Calculation of Deflection

The predicted test deflections were calculated using the variable stiffness by nonlinear analysis (*13*). The method of virtual work was used to calculate the deflections by dividing the span into 16 equal segments. The step-by-step method of calculating both the stiffness by nonlinear analysis and the deflections corresponding to each increment of load is given by Kamel (*14*). Figure 6 compares the calculated and the experimental deflection curves. The test curves represent the results of the second cycle of loading for both concrete pile tests. The second cycle is chosen to represent the behavior of the pile after the concrete is initially cracked at the first loading cycle. This assumption was used in this study and was believed to represent the case of slow cyclically laterally loaded concrete piles.

Analysis of Pile/Soil System Behavior

The LPILE computer program of Reese and Wang (*6*) was used to solve the problem of laterally loaded piles using the method of finite difference. Reese's soil *p*-*y* curves were used in the program and were included as a subroutine. The program provides deflection, moment

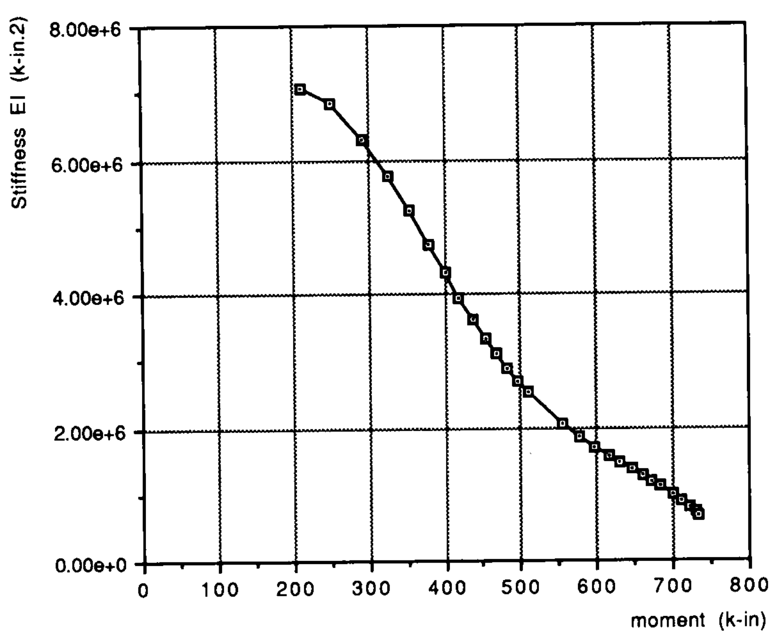


FIGURE 5 Calculated bending moment versus stiffness of 12-in. concrete pile.

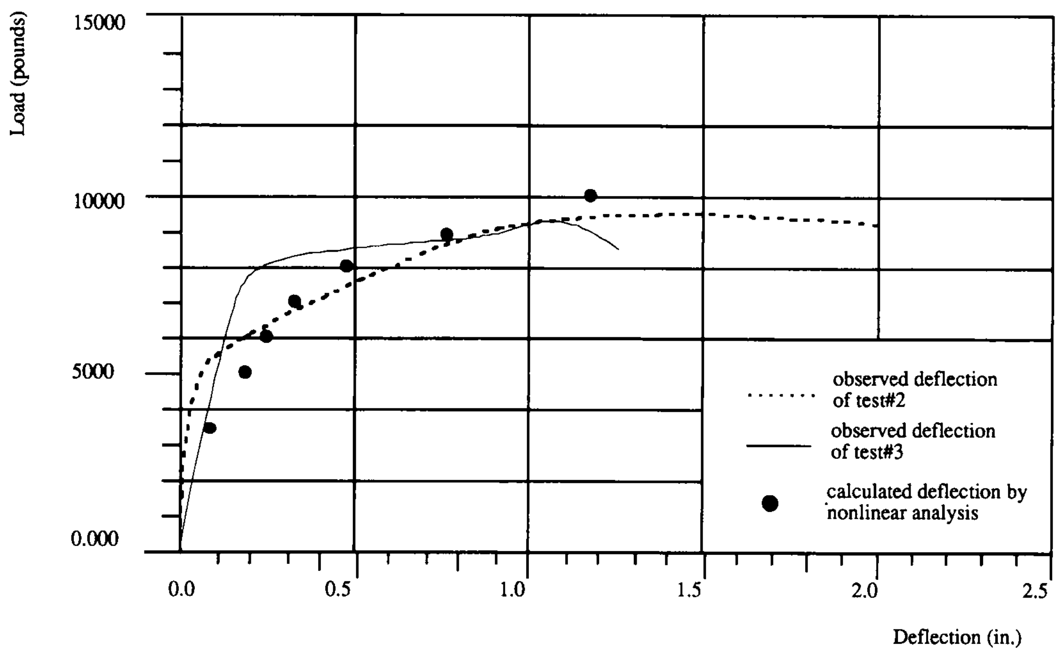


FIGURE 6 Evaluation of deflection calculation by nonlinear analysis.

diagrams, soil response, and *p-y* curves for laterally loaded piles. Reese verified the program by comparing results with a number of full-scale tests of piles under lateral loads, and these results were generally satisfactory. The program was used in this study to determine the maximum allowable horizontal deflection that a single pile can undergo without exceeding its service moment capacity.

Parametric Study

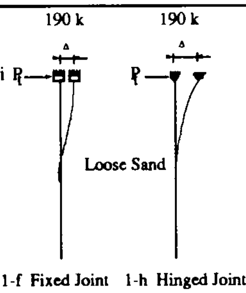
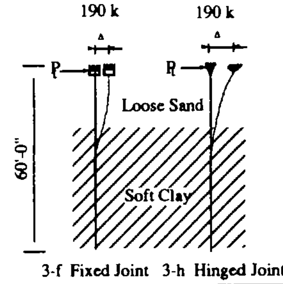
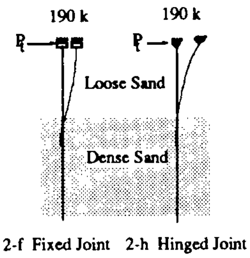
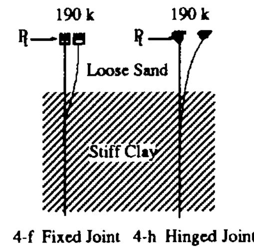
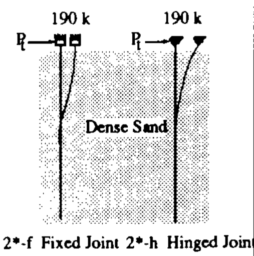
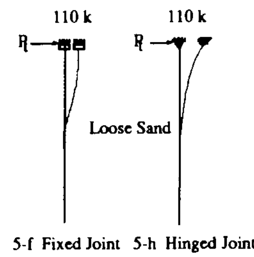
A total of 12 cases were run with the LPILE program for various types of soil and parameters. Parameters of the first five cases as an example are presented in Table 1. Cases 1, 2, 2*, 3, and 4 represent prestressed concrete piles 12 in. (305 mm) square in loose sand, loose sand followed by dense sand, dense sand, loose sand followed by soft clay, and loose sand followed by stiff clay, respectively. Cases 5 through 8 represent 10-in. (254-mm) steel H-piles with the same soils as those in Cases 1 through 4. Cases 9 through 12 represent concrete piles with a reduced modulus of elasticity of 50 percent with the same soils as those in Cases 1 through 4. The concept of using a reduced modulus of elasticity to account for creep action in laterally loaded concrete piles is being investigated by several researchers (*3,5*). Reese's *p-y* curves were used to represent soil stiffness for various types of soil.

All twelve cases were assumed to be 60 ft long (18.30 m) driven through 10 ft (3.05 m) of loose sand underlain by different types of soil, except in Case 2*. The top loose sandy layer was used to simulate the condition of using predrilled holes filled with loose sand. Case 2* was for a concrete pile driven into dense sand without a top layer of loose sand. This case is used for comparison with the case of a top loose sandy layer. In prestressed concrete piles, a constant flexural rigidity is used along the pile length. For piles subjected to large axial loads and relatively small bending moments, the section generally remains uncracked before compression stresses reach their maximum allowable limits under service loads (*14*). Both fixed and hinged pile head joints were represented in all cases. Axial loads of concrete piles were chosen to be 190 kips (845 kN) of which 100 kips (445 kN) is an effective prestressing force and 90 kips (400 kN) is applied load. Axial loads on steel piles were chosen to be 110 kips (489 kN), which represents the currently used allowable applied service load on steel piles in Nebraska.

Results of Analysis

Figures 7 and 8 show the horizontal force versus bending moment and horizontal deflection, respectively, for Cases 1, 2, and 2*. Maximum moments are determined regardless of their positions along the pile. In the case of fixed top joints, the maximum moment occurs immediately below the pile to pile cap interface. In the case of hinged joints, maximum moment occurs at a distance between 4 and 6 ft (1.2 and 1.8 m) from the pile to cap

TABLE 1 Cases of Pile/Soil Systems Run by LPILE Program

Case	Properties	Soil Layers	Case	Properties	Soil Layers
1 12" concrete pile in loose sand	pile properties and axial load concrete strength = 5.50 ksi modulus of elasticity = 4230 ksi effective prestressing force = 100 k applied axial load = 90 kips soil properties: loose sand soil modulus of subgrade = 25 pci density = 0.063 lb/in3 angle of internal friction = 30°		**3** 12" concrete pile in soft clay with 10' predrilled hole filled with loose sand	pile properties and axial load as case 1 soil properties: loose sand properties: as case 1 soft clay properties: modulus of subgrade = 30 pci density = 0.063 lb/in3 cohesion = 3.0 lb/in strain at 50% = 0.02	
2 12" concrete pile in dense sand with 10' predrilled hole filled with loose sand	pile properties and axial load as case 1 soil properties: loose sand properties as case 1 dense sand properties: modulus of subgrade = 225 pci density = 0.075 lb/in3 angle of internal friction = 40°		**4** 12" concrete pile in stiff clay with 10' predrilled hole filled with loose sand	pile properties and axial load as case 1 soil properties: loose sand properties: as case 1 stiff clay properties: modulus of subgrade = 200 pci density = 0.069 lb/in3 cohesion = 13.0 lb/in strain at 50% = 0.007	
2* 12" concrete pile in dense sand without predrilled hole	Pile Properties and Axial Load as case 1 Soil Properties: loose sand properties as case 1 dense sand properties as case 2		**5** 10x42 steel pile in loose sand	Pile Properties and Axial Load section 10x42 HP steel pile cross-sectional area = 12.4 in section modulus = 14.2 in. steel A 36 ksi soil properties: loose sand properties: as case 1	

interface. Allowable moment was calculated for concrete and steel piles on the basis of their allowable compressive stress as 335 kip-in. (37.86 kN-m) and 180 kip-in. (20.34 kN-m), respectively. The corresponding lateral loads were determined from the lateral load versus moment curves, and the corresponding lateral deflections were determined from lateral load versus lateral deflection curves. A summary of the results is presented in Table 2.

Discussion of Results

Concrete Pile Versus Steel Pile

In all cases steel piles showed more capacity to accommodate lateral deflection than concrete piles, but the difference was not very significant. For example, in loose sand a steel pile with a hinged top joint could deflect up to 0.40 in. (10.2 mm) compared with 0.34 in. (8.6 mm) for a concrete pile under the same conditions. Similar behavior was obtained under other conditions. However, in the case of concrete piles, a lateral force of 7.8 kips (34.69 kN) was needed to deflect the pile 8.6 mm (0.34 in.), whereas only 5.1 kips (22.68 kN) was needed to deflect the steel pile 0.40 in (10.1 mm). This result agreed with laboratory test results in which the steel pile showed more flexibility than concrete piles.

Effect of Predrilled Hole

The common practice of using a predrilled hole filled with loose sand was studied by introducing a layer of loose sand at the top 10 ft (3.05 m) of embedment. Table 2 and the resulting curves show that the loose sandy layer has a significant effect on the behavior of the examined cases. Because most of the deflections and moments occur within the top 10 to 12 ft (3.05 to 3.66 m) of the pile, the soil type in this region will have a significant influence on the behavior of the pile under lateral loads. This result is clearly shown in Case 2* when loose sand was not used. In this case a very small

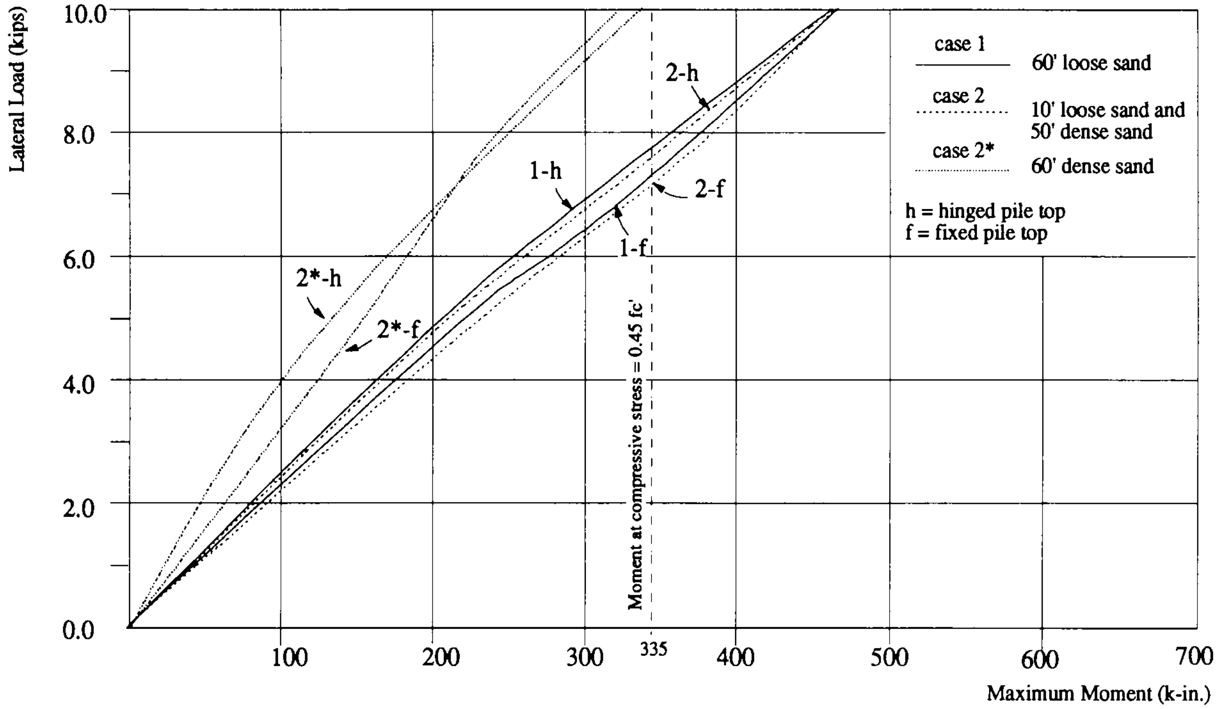


FIGURE 7 Horizontal force versus maximum bending moment for 12-in. concrete pile in sand.

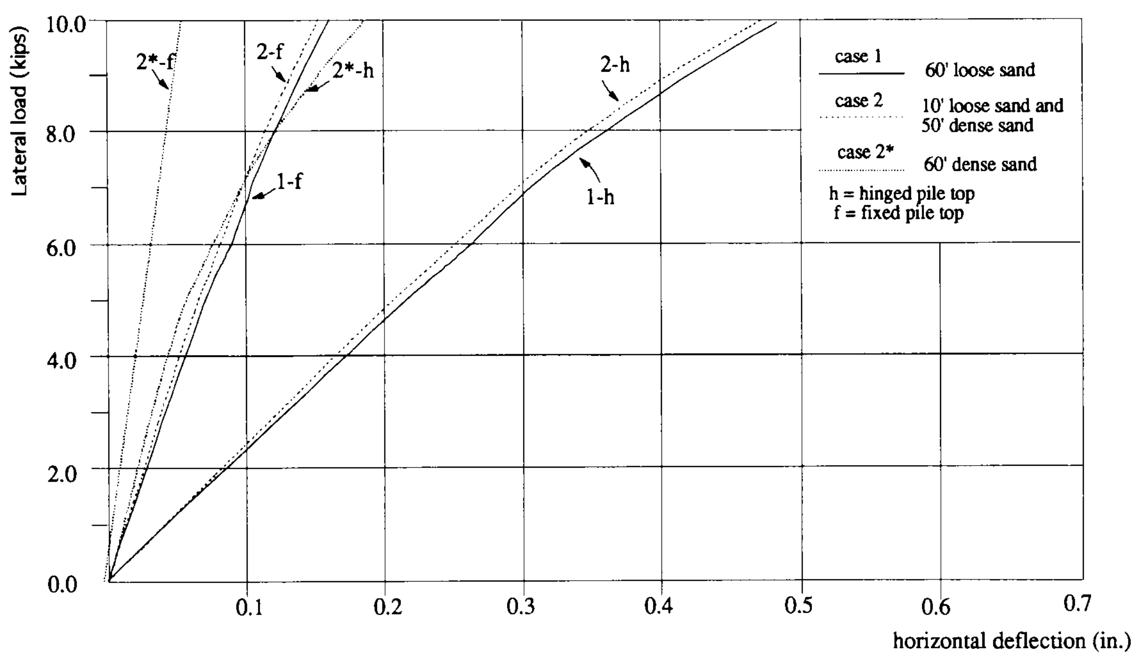


FIGURE 8 Horizontal force versus horizontal deflection for 12-in. concrete pile in sand.

TABLE 2 Summary of Analytical Results

Case	Lateral load corresponding to max. moment kip (KN)	Deflection corresponding to max. load in. (mm)	Case	Lateral load corresponding to max. moment kip (KN)	Deflection corresponding to max. load in. (mm)	Case	Lateral load corresponding to max. moment kip (KN)	Deflection corresponding to max. load in. (mm)
1-h	7.8 (34.69)	0.34 (8.6)	5-h	7.0 (31.14)	0.65(16.5)	9-h	7.8 (34.69)	0.51 (12.9)
1-f	7.2 (32.02)	0.11 (2.8)	5-f	8.0 (35.58)	0.25 (6.3)	9-f	8.1 (36.03)	0.18 (4.6))
2-h	7.8 (34.69)	0.31 (7.9)	6-h	7.0 (31.14)	0.64 (16.2)	10-h	7.8 (34.69)	0.48 (12.2)
2-f	7.2 (32.02)	0.10 (2.5)	6-f	8.0 (35.58)	0.24 (6.1)	10-f	8.1 (36.02)	0.17 (4.3)
2*-h	10.5 (46.70)	0.2 (5.1)	7-h	7.0 (31.14)	0.7 (17.8)	11-h	7.8 (34.69)	0.53 (13.5)
2*-f	10.5 (46.70)	0.05 (1.3)	7-f	8.0 (35.58)	0.22 (5.6)	11-f	8.1 (36.03)	0.18 (4.6)
3-h	7.8 (34.69)	0.34 (8.6)	8-h	7.0 (31.14)	0.65 (16.5)	12-h	7.8 (34.69)	0.51 (12.9)
3-f	7.2 (32.02)	0.12 (3.0)	8-f	8.0 (35.58)	0.21 (5.3)	12-f	8.1 (36.03)	0.18 (4.6)
4-h	7.7 (34.25)	0.32 (8.1)						
4-f	7.2 (32.02)	0.1 (2.5)	*f* =	*fixed joint*		*h* =	*hinged joint*	

amount of deflection was sufficient to cause the maximum allowable lateral load. In addition, very close results were obtained in all cases when using an upper 10-ft (3.05-m) loose sand layer, regardless of the type of soil below this depth.

Effect of Reduced Modulus of Elasticity

Reducing the modulus of elasticity has a significant effect on the deflection of the concrete piles. In Cases 9 through 12, maximum deflections were increased by about 50 percent when using a reduced modulus of elasticity.

Proposed Pile Abutment Joint

The feasibility of using a sliding joint for pile/abutment connection was investigated. A joint capable of allowing the abutment to slide and rotate over the top of the pile would allow for more lateral expansion than would a rotationally restrained pile top connection. Figure 9 shows a proposed joint detail. A bearing pad was used at the top of the pile. The pad consists of a layer of random-oriented reinforced-fiber neoprene coated with a Teflon layer. The Teflon layer allows lateral movement against an embedded steel plate that is connected to the cast-in-place concrete abutment by welded studs or rebars. The four sides of the pile top were covered by compressible material such as expanded polystyrene or urethane Styrofoam. The compressible material at the two sides, in the direction of deflection, allows lateral movements. On the other two faces (front and rear) the compressible material was used to break the bond between the abutment poured concrete and the pile head when the abutment moves laterally. The joint could be manufactured in one piece and placed on the pile top once the pile is driven to its final position. In cases where the pile top is damaged as a result of driving, the damaged part should be either cut or cast with rich cement grout to keep the original dimensions of the pile. If the damage is not serious, the pile top may not require further treatment before placing the joint.

Testing of Proposed Joint

A prototype of pile abutment joint was tested in the laboratory to verify its behavior. The bearing elastomer pad should have the following capabilities:

1. Carry an applied axial load of the pile, which is about 90 kips (400 kN) for a 12-in. (305-mm) concrete pile,
2. Allow some rotation between upper and lower faces of the pad; and
3. Have minimum coefficient of friction to allow sliding against the upper steel plate.

There are many bearing pad producers, and each one provides its own design guides. Any bearing pad that meets the above requirements could be used. The chosen pad for the test was manufactured by JVI, Inc., and designed according to their Design Guide Handbook (7). A sliding pad was chosen with a thickness of 1/2 in. (12.7 mm) and a dimension of 8 × 8 in. (203 × 203 mm). The JVI "MASTICORD" pad was coated with a Teflon layer 3/32 in. (2.4 mm) thick. The pad has an allowable compressive stress of 2.50 ksi (17.23 Mpa) and a coefficient of friction of less than 5 percent against smooth stainless steel surface (2B or mirror finish (7)).

Figure 10 shows the schematic test setup as well as the test results. Vertical loads as well as lateral loads were applied to the joint. Lateral deflections were measured for various lateral loads acting under constant

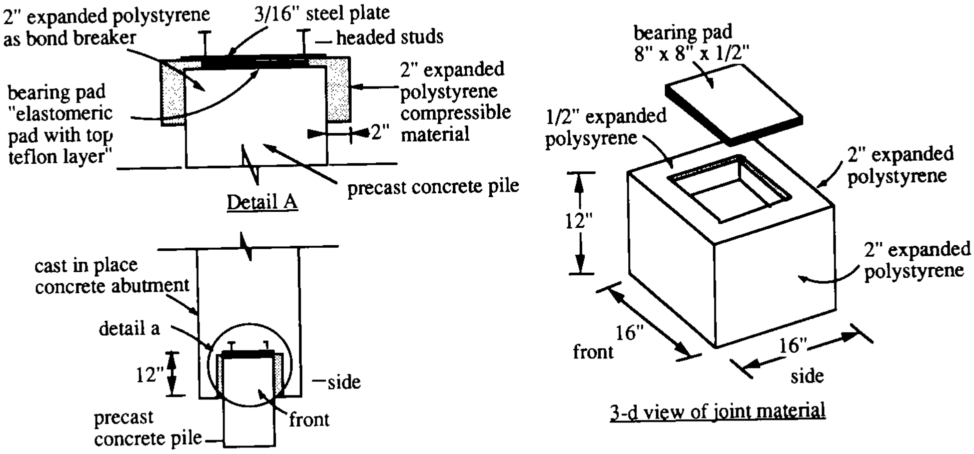


FIGURE 9 Proposed concrete pile abutment joint.

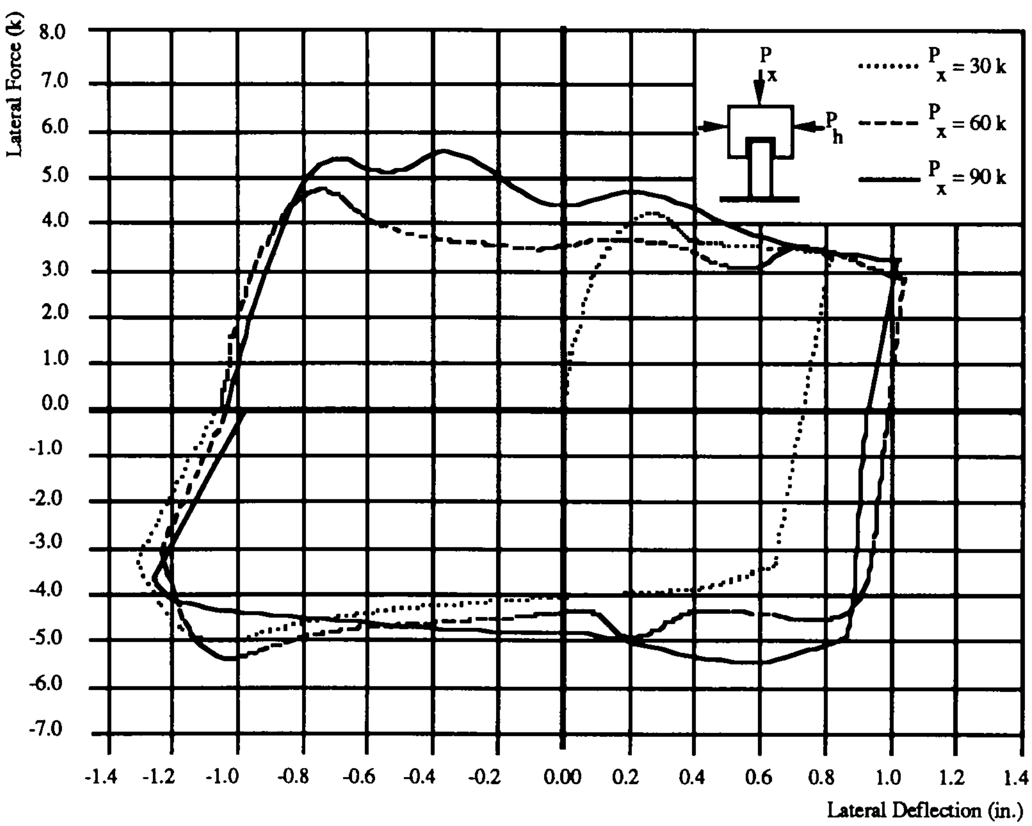


FIGURE 10 Test results of the proposed pile to pile cap joint.

vertical loads. The vertical loads were 30, 60, and 90 kips (133, 266, and 400 kN). Lateral loads were applied until the cap moved against the pile head unit for about 1 in. (25.4 mm); then the load was reversed.

The results show the capability of the proposed joint to allow lateral movements of about 1 in. (25.4 mm) in each direction under sustained vertical loads up to 90 kips (400 kN). The lateral force that overcomes the friction and the resistance of the compressible material was about 5 kips (22.3 kN). The observed maximum vertical deflections caused by the compressibility of the joint system were 0.1, 0.17, and 0.2 in. (2.5, 4.3, 5 mm), corresponding to 30, 60, and 90 kips (133, 266, and 400 kN) vertical loads.

Longitudinal Stability

According to AASHTO (*15*), braking force is estimated as 5 percent of the bridge live load. In bridges, live load normally does not exceed 40 percent of the total vertical load. Therefore, the maximum horizontal longitudinal breaking force on the pile is estimated to be 1.8 kips (8.0 kN). Testing of the proposed joint showed that it could resist up to 5 kips (22 kN) before sliding occurs.

Cost Analysis

The total material cost of the proposed joint was $25. The cost of the bearing pad was $18, and the rest of the cost was for the steel plate and the compressible material. This cost is equivalent to the approximate cost of 2 ft (0.6 m) of pile. When the proposed joint is used, it will not be necessary to use the predrilled holes filled with loose sand unless they are needed for other purposes.

Conclusion

The detail of the pile abutment connection that is currently used in Nebraska and other states for steel piles needs to be modified for possible use with concrete piles. Allowable lateral deflections of concrete piles with rotationally restrained pile abutment joints would be small; hence, the use of such joints with concrete piles would be limited to short integral abutment bridges.

Using the uncracked stiffness properties for prestressed concrete piles is a proper assumption for calculating lateral deflections of a fully loaded pile. Attempting to take advantage of a reduced cracked section stiffness in calculating lateral deflections is not an appropriate assumption.

The 10-in. (254-mm) HP steel pile is more flexible than the prestressed 12-in. (305-mm) concrete pile. The capacity of steel piles to accommodate lateral deflections is greater than that of concrete piles. However, the difference is not significant when compression stresses are limited to their allowable values.

The common practice of using a predrilled hole filled with loose sand has a significant effect on the behavior of laterally loaded piles. Because most of the deflections and moments occur within the top 10 ft (3.05 m) of the pile, soil type in this region will always control the behavior of the pile regardless of the type of soil below this depth.

A new type of joint is proposed for use at the top of concrete piles. The joint allows relative lateral movement up to 1 in. (25.4 mm), with a maximum generated lateral load of 5 kips (135 kN). In most soil types the concrete piles deflect elastically about 0.15 in. (3.8 mm) under this lateral load. The proposed joint would allow the prestressed concrete piles to be used in integral abutment bridges with a total allowable movement of at least 2.3 in. (58.4 mm). Therefore, concrete piles could provide an alternative design solution in integral abutment bridges where steel piles are currently used.

ACKNOWLEDGMENTS

The authors wish to thank the Nebraska Department of Roads, the Precast Concrete Association of Nebraska, and the Center for Infrastructure Research, University of Nebraska, for sponsoring this project. Special thanks are due to Morrie Workman of the Wilson Concrete Company and his staff for their assistance during the pile testing. Thanks go to Amin Einea for his valuable input in reviewing this paper. Deborah Derrick was helpful in proofreading and providing editorial input.

REFERENCES

1. Greimann, L. F., R. E. Abendroth, D. E. Johnson, and P. B. Ebner. *Pile Design and Tests for Integral Abutment Bridges*. Final Report. Iowa DOT Project HR-273, ISU-ERI-Ames 88060. Dec. 1987.
2. Greimann, L. F., D. D. Girton, and T. R. Hawkinson. *Validation of Design Recommendations for Integral Abutment Piles*. Report. Iowa DOT Project HR-292, Ames, Sept. 1989.
3. Wasserman, E. P. Jointless Bridge Decks. *Engineering Journal*, American Institute of Steel Construction, 3rd quarter, 1987.
4. Evans, L. T., Jr., and J. M. Duncan. *Simplified Analysis of Laterally Loaded Piles*. Report UCB-GT/82-04. De-

partment of Civil Engineering, University of California, Berkeley, July 1982.
5. Reese, L. C. Behavior of Piles and Pile Groups Under Lateral Load. Research Report. FHWA/RD-85/106. U.S. Department of Transportation, Washington, D.C., 1985.
6. Reese, L. C., and S.-T. Wang. *Documentation of Computer Program LPILE, Version 3*. University of Texas, Austin, 1989.
7. *Masticord Structural Bearing Pad, Design Guide*, 3rd ed. JVI Inc., Skokie, Ill.
8. Sheppard, D. A. Seismic Design of Prestressed Concrete Piling. *PCI Journal*, Vol. 28, No. 2, March–April 1983.
9. Pam, H. J., and R. Park. Flexural Strength and Ductility Analysis of Spirally Reinforced Prestressed Concrete Piles. *PCI Journal*, Vol. 35, No. 4, pp. 64–83, July–August 1990.
10. Hognestad, E. *A Study of Combined Bending and Axial Load in Reinforced Concrete Members*. Bulletin 399. University of Illinois Engineering Experiment Station, Urbana, Nov. 1951.
11. Lin, T. Y. *Design of Prestressed Concrete Structures*, 3rd ed., p. 5.29, 1981.
12. Devalapura, R., and M. K. Tadros. Stress-Strain Modeling of 270 ksi Low-Relaxation Prestressing Strands. *PCI Journal*, Vol. 37, No. 2, March–April 1992.
13. Tadros, M. K. Expedient Service Load Analysis of Cracked Prestressed Concrete Sections. *PCI Journal*, Vol. 27, No. 6, Nov.–Dec. 1982.
14. Kamel, M. Precast Prestressed Concrete Piles in Integral Abutment Bridges. Master's thesis. University of Nebraska, 1992.
15. *Standard Specifications for Highway Bridges*, 13th ed. AASHTO, Washington, D.C., 1983.