# Commentary Appendix for Technical Documents

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## Pile bents with steel H-piles ~ 22 June 2009

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**Summary**

The original 1950s office computations for the H-piles on the P10A sheet (Standard Design Concrete and Steel Piles…for Use in Trestle Pile Bents – P10A) have been lost. In the 1990s it was believed that for the original computations fixity was assumed 6 feet below ground or streambed. Computations from the 1990s also assumed that pile heads were pinned, that K = 0.80, and that axial load was applied at an eccentricity of one inch. Those assumptions generally need to be updated.

Recent research has indicated that an H-pile with an embedment in the cap of 12 inches, the minimum on the P10A sheet, will be at least partially fixed. Larger embedments of 1.5 to 2.0 times the pile size result in nearly complete fixity.

Fixity of an HP-10 or HP-12 at 6 feet in soil is about the same as the results from the 1960s Davisson method, but the method is believed to be unconservative and no longer is permitted for final design in the AASHTO LRFD Specifications. The pile design method of choice now is the p-y method, which is programmed in LPILE (formerly COM624P). Based on runs of LPILE with H-piles in soft clay, the slenderness for column design, KL/r, should be based on $K \geq 0.90$ if the pile fixity is determined by the Davisson method. Thus those LPILE runs indicated that the Davisson method is unconservative with respect to the past office assumption.

Based on preliminary studies of piles with loads at a 2-inch eccentricity, a general study including runs of strip frames in STAAD and runs of LPILE (that determined adjustments for the Davisson method), and a revised general study, the following table for the P10L sheet was developed. For H-piles the service limit state (plan sheet bearing) load values remain the same as on the P10A sheet, but the permissible heights (H) are adjusted for more accurate column slenderness standards. With addition of HP-14 piles to the P10L sheet the nominal resistances will be sufficient for the pile bents for the J-series and H-series bridges and the present maximum heights on the P10A sheet. In some cases, reduced maximum heights on the P10L sheet will require larger H-pile sections for a pile bent.

**Permissible heights, nominal resistances, and maximum plan sheet bearing for Grade 50 H-piles in pile bents**

| H-pile section | Maximum height, H (1), feet | | Recommended minimum height, H (2), feet | Minimum soil penetration (3) feet | Nominal Resistance, $P_n$ kips (4) | Max. plan sheet bearing, tons |
|---|---|---|---|---|---|---|
| | Monolithic cap | Non-monolithic cap | | | | |
| HP 10x42 | 19 | 15 | 7 | 18 | 154 | 37 |
| HP 10x57 | 19 | 16 | 7 | 18 | 208 | 50 |
| HP 12x53 | 23 | 20 | 9 | 21 | 192 | 46 |
| HP 14x73 | 28 | 25 | 13 | 24 | 265 | 64 |
| HP 14x89 | 29 | 26 | 13 | 24 | 324 | 78 |

Table notes:

(1) H is defined as the height from underside of pile bent cap to ground line or stream bed (with scour, if applicable).
(2) As the pile becomes shorter it becomes stiffer and attracts more moment but has greater axial capacity. Consult with the supervising Unit Leader before using a lesser height.
(3) This distance from ground line or stream bed to bottom of pile is required for the pile to have a point of fixity. If the pile has less embedment it may fail by "fence-post" action.
(4) These nominal resistances are intended to be used with $\varphi_c = 0.70$. These values at Structural Resistance Level - 1 (SRL-1) were calculated from 1.45*6 ksi*A/0.7, which simplifies to 12.43*A (and then were converted to metric).

Since the P10A sheet was developed, AASHTO has added to the minimum pile spacing rule. The original minimum spacing of 2.5 feet now only is applicable for piles 12 inches or less in size. With the additional AASHTO 2.5D rule, for a 14-inch pile the minimum spacing is 2.92 feet. In some cases, the J-series and H-series pile spacings are less and therefore do not follow the AASHTO rule.

The AASHTO LRFD Specifications no longer specify minimum pile penetration: 10 feet in hard cohesive or dense granular material, 20 feet in soft cohesive or loose granular material, or one-third of the unsupported height (H) of a pile in a pile bent. In most cases, however, the minimum penetration in the table above is a more stringent limitation, unless the designer specially analyzes the pile.

**History**

A P10 sheet, dated 1953 and based on the 1949 AASHO Standard Specifications, predates the earliest P10A sheet in the archive of drawing files. The P10 sheet was intended for several continuous concrete slab (CCS), pretensioned prestressed concrete beam (PPCB), and rolled steel beam (RSB) standard plan bridges. The sheet shows four pile types and requires driving to full penetration but also to a minimum 30-ton capacity.

The first P10A sheet was issued in 1959 and had five trestle pile types (plus four foundation pile types) including the three on the present P10A sheet. Bearing values and maximum height (H) dimensions on the present sheet are the same as they were on the 1959 sheet. For P10A, H-piles the bearing value is based on 6 ksi, which was the limit for point bearing piles in the 1949 or 1957 AASHO Standard Specifications. At that time the usual steel specification was A7 with a minimum yield of 33 ksi.

In the 1949 and 1957 AASHO Standard Specifications, presumptive bearing values in tons for steel and concrete friction piles generally were set in tons at pile size in inches times two. Also in the 1957 AASHO Standard Specifications are the following:

- Piles shall be designed as structural columns [1.4.17].
- A Converse-Labarre formula is suggested for friction pile load reduction due to group effects [1.4.17]. (It does not appear that this rule was used for P10A piles.)
- Concrete piles shall be designed in accordance with article 1.4.11 (referenced from 1.4.17), which refers to 1.7.8 for columns, which generally follows the 1951 ACI Code.

- …steel piles in accordance with article 1.4.2 (referenced from 1.4.17), which gives rules, but which also permits use of Appendix B. Appendix B is required for compression members with known eccentricity. The rules for concentrically loaded columns with L/r not greater than 140 distinguish between riveted ends and pinned ends, having deductions from a basic stress for $(L/r)^2$ without accounting for sway or end effects. In Appendix B, the L is noted as being 75% of the total length with riveted connections and 87.5% of the total length for pinned connections. In 1.6.9 main compression members are limited to an unsupported length to radius of gyration ratio of 120.
- …concrete filled pipe piles in accordance with article 1.4.11 (referenced from 1.4.17), except that allowable unit stresses may be increased 20% provided the shell thickness is not less than ¼ inch. The area of the shell shall be included in determining the value of p, percentage of reinforcement. Where corrosion may be expected, 1/16 inch shall be deducted from the shell thickness….
- The allowable stresses of articles 1.4.2, 1.4.11, and 1.4.14 may be used in all cases where all of the stresses to which the piles may be subjected have been included. These stresses may be increased in accordance with article 1.4.1 (referenced from 1.4.17), which gives the various load groups and "percentages of unit stress" that are greater than 100% for load groups including wind, temperature, earthquake, and/or ice.
- For trestle piles…designed for dead load and live load only and where temperature, traction, water pressure and other forces are not considered, the allowable unit stresses…shall be decreased 20%. (This rule also was in the 1949 AASHO Standard Specifications and appeared in all specifications through 1977.)
- The 1957 AASHO Standard Specifications do not cover prestressed concrete because of rapid developments and lack of consensus [page XXIII]. Prestressed concrete was added to the 1961 AASHO Standard Specifications.
- The use of KL/r for steel compression members (including the effective length factor, K) first appeared in the 1977 AASHTO Standard Specifications.

Maximum height values on the P10A sheet favor larger piles at more than a simple ratio of radii of gyration. For example, the $r_y$ ratio for HP 12x53 to HP 10x42 is 1.19, but the ratio of maximum H-values, 26/16 is 1.63. Assuming the piles are 6 feet longer to a fixed connection in the soil does not resolve the difference because 32/22 is 1.45. Obviously something other than a simple L/r ratio for axial compression was used to determine the height values on the original P10 or P10A sheets.

Recently there have been questions regarding design of the piles for eccentricity and design of the cap for a failed pile. In the mid-1990s notebooks from which the electronic manual was prepared is a 1968 handwritten pier design summary sheet that states the following for pile bent piers.

> Pile bent piers (P10A type) are to be designed for no horizontal load. Piles are assumed flexible in two directions. However, the Pec/I of eccentric vertical loads must be considered on outside piles. (The Pec/I referred to placing traffic lanes off center to determine the maximum pile loads.)

The last sentence is circled and noted "No. Use P/A all loads." Guidelines in the electronic manual had no provision for eccentric loads, and the present design manual generally follows the same direction with a statement that piles need not be checked for accidental eccentricity (intended to mean that field tolerances need not be considered).

An old design summary sheet headed "Iowa State Highway Commission Bridge Department" addressed to a design squad states the following as changes to previous practice:

1. Use impact in determining number of piles required. (per "Bridge Design Department Design Criteria for Piers.")
2. Use straight P/A for all vertical loads—do not account for any eccentricity.
3. Check pile cap reinforcing allowing for damage to piles and using operating stresses.
    a. End pile removed—check cap as cantilever.
    b. Interior pile removed—check cap as a restrained beam.

Undated (but with 1960 concrete placement diagrams) computations for H-12 and H-13 standard bridges show pile reactions computed using P/A + Pec/I for live loads in off-center lanes and identifies the procedure as "P.F. Barnard's method." Based on the various written sources there have been two procedures followed in the office for determining live loads: (1) use P/A + Pec/I for eccentric lanes and (2) use P/A for maximum live load. The first procedure generally will give a slightly larger outer pile load, and that load then would be used for all piles.

To deal with potential eccentricity in individual columns due to tolerances, codes have taken various approaches, and those approaches do not appear to be in the 1949 or 1957 AASHO Standard Specifications.

Development of the AISC (steel) allowable stress specification included some provision for out-of-straightness in the allowable axial compression stress equation, but no minimum design eccentricity was specified. Many typical connections have measurable eccentricity that can be used in design.

The ACI (concrete) Building Code has dealt with the issue differently. In the 1963 and 1971 codes a minimum design eccentricity of 0.1 times the tied column dimension was required for strength design, perhaps for formwork tolerances. Later editions of the ACI Code eliminated the minimum eccentricity rule by simply topping-off the maximum column strength with a multiplier of 0.8 for tied columns. For evaluation of slenderness effects, editions of the code since 1977 have specified that a minimum end moment be used based on an eccentricity in inches of 0.6 + 0.03h.

For piles a minimum design eccentricity is dependent on the relatively large tolerance in pile placement. Except for prestressed concrete piles the Iowa DOT requires only that piles be placed within 3 inches of intended location at start of driving. In some cases, pile bent H-piles are flexed after driving to align relatively well at the cap, but some deviation from the correct location remains at the ground level. In other cases, H-piles are not flexed, and piles are 3 inches or more out of position at the cap.

The rules for design of the P10A cap for a failed pile (which follow a probably earlier document) were on the 1968 design summary sheet as follows:

- Check P10A cap for pile failure using operating stresses.
    (a) End pile removed
    (b) Interior pile removed.

These rules have been retained in the present Bridge Design Manual and are being included in the LRFD Bridge Design Manual with a specified load factor and the HL-93 load.

The P10A sheet requires 1'-0 pile embedment with dowels for monolithic caps and 1'-6 embedment without dowels for non-monolithic caps. For the continuous concrete slab (CCS) J-06-series the monolithic cap is dimensioned 1'-0 deep, and thus the pile embedment does not cause piles to project into the slab. For both J-06- and pretensioned prestressed concrete beam (PPCB) H-06-series, non-monolithic caps are dimensioned for a three-foot square cross section, which causes the piles to project to mid-height of the cap.

Original computations for 1960 H12 and H13 PPCB bridge standards by someone in the office show only dead, live, and impact loads being applied concentrically to pile bent piles. The plans refer to the P10A sheet and give pile bearing and number of H-piles based on 16-foot and 26-foot heights from stream bed to bottom of cap. These pile heights match the steel H-pile conditions on P10A but not the 13 and 22-foot maximum heights for steel pipe and prestressed concrete piles. Probably it was expected that the user of the standards would make appropriate choices that would not conflict with either the H-standards or sheet P10A.

In the mid-1990s Nucor-Yamato often rolled HP 12x53 piles with reduced area. About 1997 in the office John Harkin checked the piles for P10A conditions. He assumed encasement was non-structural, fixity at 6 feet below ground (presumed to be the original design condition), K = 0.8 for fixed base and braced, pinned head conditions, and design eccentricity of one inch about each axis. For battered piles he assumed the lateral component of load to be taken by the superstructure and assumed one inch eccentricity for the axial load. His computations indicated for full cross sections the following:

- 46 tons allowable at 26 feet for HP 12x53 (which matches the P10A sheet) and
- 37 tons allowable at 19 feet for HP 10x42.

The HP 10x42 height is 3 feet more than shown on the P10A sheet. For HP 12x53 piles with 6% section loss the permissible load at 26 feet was reduced to 43 tons.

The present office procedure guide contains the following examples:

- A Mathcad sheet for a CCS bridge that designs piles based on P/A, using strip live load extended over the entire roadway.
- A Mathcad sheet for a CCS bridge that assumes piles are fixed 6 feet below ground, one-inch eccentricity about both axes, K = 0.8, and P/A + Pec/I.
- Hand computations for a PPCB bridge that design piles for P/A + Mc/I assuming either two or three lanes are loaded, thereby considering eccentric traffic.

**Issues**

- **Limit states: For strip frame analysis for a J-series or H-series bridge** consider only the Strength I limit state. Strength III and V, which include wind, are not likely to control because the jointless superstructure acting as a wide horizontal beam will carry all except part of the substructure wind loads to the abutments. The superstructure also will carry braking forces to the abutments. Thus the loads in Strength I to consider are dead (DC and DW), live and dynamic load allowance (LL and IM), water (WA), and temperature (TU).

  Consider the Extreme Event limit state with respect to scour by considering pile length to scour elevation.

- **Loading:** Although office practice has been to neglect weight of individual pile encasement, the dead load of the encasement, especially for HP-14 piles, is large enough that it was included in the general study and should be included in future designs.

  Even though the pile cap is relatively rigid, individual H-06-series beam loads can cause the pile axial loads to vary plus or minus 20 to 25%, if the diaphragm at the bent between the beams is neglected. If the diaphragm is considered, the variation in axial loads will be less. The office practice of checking the cap for a missing interior or end pile ensures that the cap is adequate for reasonable variations in loads.

  Because pile bents are intended to be used with relatively small watersheds, water loads will be small. A water load on a tall pile generally will result in relatively small moments at the top of the pile where a strip frame analysis indicates the largest moment for a fixed-head pile. The revised general study considered a small water load indirectly by considering moment magnification as if a load between braced points existed.

  A one-inch temperature movement has relatively large effects on a 45-degree skewed non-monolithic pile bent. For the maximum H-06-series and J-06-series bridge lengths, the movement for a steel substructure designed by LRFD will be less than one-half inch.

  Moments caused by temperature movement can be estimated quickly and accurately using fixed end moment formulas for either a pinned top/fixed base or a fixed top/fixed base pile.

  Moments caused by dead, live, and impact loads cannot be estimated accurately by simple formulas. Reasonable office practice for typical bridges would be to estimate moments using a two-dimensional frame model including integral abutment piles, pile bent piles, and a superstructure strip of appropriate width and stiffness. Although the frame could be analyzed by moment distribution it can be analyzed more efficiently by STAAD or similar finite element software.

- **Pile head bracing**: Typical Iowa pile bent bridges have jointless decks and integral abutments. The horizontally rigid deck can convey support from abutments and wing walls to pile bents. Therefore, in completed bridges, piles are laterally braced at their heads, and loads transverse to the bridge are carried mostly by the superstructure acting as a deep horizontal beam.

  J-06- and H-06-series bridges are quite rigid in the horizontal plane. For a J24, 150-foot bridge, deflection under full wind load on superstructure and live load with supports only at abutments was computed as 0.017 inches at midspan of the center span. At pile bents, deflection will be smaller, and for wider bridges deflection will be smaller. For longer bridges, however, deflection will be greater. AASHTO-LRFD

4.5.3.2.2 allows neglecting sway moment magnification if $\Delta \leq l_u/1500$, which computes to 0.056 inches for a 7-foot pile height and 0.128 inches for a 16-foot pile height. These computations verify that pile heads can be considered braced.

During construction, after the cap is poured and cured, battered piles provide lateral support in the plane of the bent. Out-of-plane, however, the bent has no lateral support other than any construction bracing.

- **Fixity at tops of piles embedded in monolithic or non-monolithic caps:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require an embedment of 12.0 inches minimum or a reduced embedment of 6.0 inches minimum with embedded dowel, bar, or strand attachment. The P10A sheet requires 12-inch embedment with dowels for piles in monolithic caps (J-06-series bridges) and 18-inch embedment without dowels for non-monolithic caps (J-06- or H-06-series bridges). The P10A sheet requirements exceed the AASHTO minimums.

  A group of researchers determined 61 to 83% fixity for one-diameter pile embedment [Castilla et al. 1984]. By computation, Tennessee engineers [Wasserman and Walker 1996] showed that a 12-inch embedment (1.2-diameter) of an HP 10x42 could develop the full, strong-axis plastic moment. Another group of researchers assumed 50% fixity for one-foot-embedment (1.2-diameter) of HP 10x42 piles [Hughes et al. 2007]. For full fixity at a pile cap, the Kansas design manual [KDOT 2007] recommends a minimum of 1.5-diameter embedment. In a literature review, researchers [Rollins and Stenlund 2008] reported that testing by others had determined an HP 10x42 embedded 12 inches (1.2-diameter) developed the full moment capacity of the pile about the weak axis but not about the strong axis. They also reported from other testing that an HP 14x89 embedded 5 inches (0.36-diameter) developed 25 to 66 percent of the ultimate moment capacity.

  Based on the various tests and studies conducted to date, the P10A connections for steel H-piles may be classified as partially fixed at the monolithic J-06-series cap and fully fixed to the J-06-series and H-06-series non-monolithic cap.

- **Fixity at non-monolithic cap to superstructure connection:** Possiel conducted tests of a prestressed beam to bent cap connection made with two elastomeric pads (similar to the H-06-series connection, except without the keyed-in diaphragm) and found that the connection resulted in partial fixity [Possiel 2008]. Based on this testing and analysis, the J-06- and H-06-series non-monolithic cap to superstructure connection, out-of-plane, may be classified as partially fixed. In the plane of the pile bent, where the full width of the superstructure is engaged, the connection should be considered fixed.

- **Fixity at bottoms of piles:** The AASHTO LRFD Specifications give an approximate method to determine point of pile fixity [AASHTO-LRFD 10.7.3.13.4] developed in the 1960s by Davisson. At this time the method is known to be unconservative to some [Wilson et al. 2005] but not to all [Kumar et al. 2007]. Although the method was permitted for final design in the 2nd Edition of the AASHTO LRFD Specifications, in the 3rd and 4th Editions the method is be used for preliminary design only.

  In developing a model for analysis of a pile subjected to seismic loads, Chai noted that the depth to fixity based on lateral deflection typically is on the range of three to six pile diameters [Chai 2002]. (The 6 feet used for HP-10 piles in the P10L revised general study is 7.2 diameters.) Chai also indicated that the actual maximum moment in the pile would develop above the depth to fixity.

  A more accurate point of fixity based on p-y analysis was developed for integral abutment piles [Greimann et al. 1987], and it was shown to vary for lateral load, moment, and buckling. The most recent method to estimate point of fixity and analyze driven piles is to use LPILE or comparable software that utilizes the p-y method.

  Russo also noted that the point of fixity will vary for different pile loading conditions but that a single “average point of fixity” works well for many different load conditions [Russo 1999].

With LPILE (previously COM624P), determining the point of fixity seems to be open to engineering judgment. In an integral abutment example, engineers at the Tennessee DOT took the location of first zero deflection below ground as the fixed point [Wasserman and Walker 1996], whereas for pile bents, engineers at the North Carolina DOT take the fixed point as either the maximum negative deflection or the maximum negative moment below ground [Possiel 2008].

The point of fixity will vary depending on the stiffness and head condition of a pile [Possiel 2008]. The location will be deeper for a stiffer pile. It will be deeper for a fully fixed pile head rather than a pinned head, but the depth will depend on whether it is determined as first zero deflection, maximum negative deflection, or maximum negative moment.

The point of fixity also will vary depending on the characteristics of the soil. For the general study, soils were assumed to have a minimum Standard Penetration Test $N \geq 3$ for cohesive soils and $N \geq 5$ for granular soils. These N-values correlate only approximately with general characterizations of soils [Teng 1962].

For the pile to have a point of fixity the pile must penetrate into the soil sufficiently so that there are at least two points of zero deflection below ground [Wang and Reese 1993]. If the pile does not penetrate far enough the bottom of the pile will deflect laterally in a “fence-posting” mode of behavior.

- **Buckling length and K-value**: Based on the P10A sheet, office practice has been to encase H-piles from the cap to 3 feet below ground or streambed. Because there is no positive shear connection this encasement is not considered to be composite with the column and has been neglected in office computations.

  However, construction of the encasement will initially disturb the upper 3 feet of soil. Considering the potential disturbance, for preliminary studies of buckling length the 3 feet was added to the depth to fixity determined by the Davisson method, which was reduced by one foot based on Alabama research [Hughes et al. 2007].

  For the revised general study the depth to fixity was simplified to neglect the effects of encasement. The soil disturbance during construction due to the encasement will increase the depth to fixity, but the effect of the disturbance will dissipate during service. Even though the encasement is not composite it will stiffen the pile and generally compensate for the increased depth to fixity.

  With an LPILE analysis the buckling length can be determined as the distance between points of zero moment, which represent inflection points [Wasserman and Walker 1996]. For a pinned-head pile this distance is from the pile head to the first point of zero moment on the pile below. For a fixed-head pile, one distance is between the first two points of zero moment on the pile [Wasserman and Walker 1996], but there also is a distance above the first point that should be doubled to make a full buckling curve. This upper distance seems to be recognized but incorrectly analyzed in the literature [Wang and Reese 1993] and is difficult to determine if the pile head is not held to a fully fixed head zero slope.

  When using the buckling lengths determined from LPILE, the K-values will be 1.0 because the lengths are for full buckling curves. When the lengths are related back to a pile modeled with a length set by superstructure and streambed or ground elevations, however, the Ks will vary.

  For the general study, after comparing buckling lengths determined using the Davisson method for point of fixity and LPILE, it appeared that establishing fixity with the Davisson method and using $K = 0.90$ for fixed head and $K = 0.95$ for pinned head piles approximated the results from LPILE. Because of the simplicity of the Davisson method it gave more consistent results than LPILE and, the Davisson method with the K-values above was used for the general study. However, the heights determined for the P10L sheet were conservative in some cases. To remove the conservatism for H-piles, in the revised general study specific K-values were determined from LPILE for each-pile size and for each axis, strong and weak.

  For primary steel members the AASHTO LRFD Specifications have a slenderness limit, $KL/r$, of 120 [AASHTO-LRFD 6.9.3], and this limit was followed in the general study.

- **Minimum pile spacing:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require that piles be spaced center-to-center not less than 30.0 inches or 2.5 pile diameters, whichever is larger. (The two-part rule has been in the AASHTO Standard Specifications since the late 1980s or early 1990s.) Based on width, for 14-inch H-piles the minimum spacing is 35 inches or 2.92 feet. For J24-06 series bridges 120 feet or more in length and skewed 15 degrees or less, pile bent piles are spaced too close to meet the minimum spacing requirements for a 14-inch H-pile. For H40-06 series bridges piles are spaced adequately.

- **Eccentricity:** Although the Iowa DOT Standard Specifications allow a pile to be placed for driving up to 3 inches from its intended location, piles generally are driven plumb and are unlikely to lean unless driven past boulders or other obstructions. Contractors generally do not use templates for pile positioning unless placement is more important than usual, such as for a precast cap or wall encasement. H-piles may be flexed into cap alignment or may be up to 3 inches or more from their intended head locations. Construction staff generally does not permit bending sufficient to yield piles [Kyle Frame, 12 May 2008].

  Generally there are no requirements for minimum eccentricity for steel H-piles, although some office computations have adopted arbitrary values or values from the ACI Code for concrete.

  Various experiments with pile bents analyzed by STAAD have shown little effect of eccentricity when piles are connected with the usual concrete cap. Because of pile cap stiffness and continuity, misplacing a pile 2 inches along the cap will have little effect on pile loads. Misplacing a pile 2 inches transverse to the cap will cause all piles for a 24-foot roadway bridge to share the eccentric moment approximately equally.

- **Battered piles: Present office practice is to** assume that increase in axial load in battered piles may be neglected. Several preliminary pile bent STAAD runs indicated that the axial load increase would be in the 3% to 9% range.

- **Scour:** Even though pile bents are to be used only for sites with small streams and limited drainage area, some sites may be subjected to significant scour. In those cases the designer should consider the stream bed to be at the maximum scour elevation and choose a pile size and shape for the increased height (H).

- **Pile penetration:** The 2$^{nd}$ Edition of the AASHTO LRFD Specifications required that piles penetrate a minimum of 10 feet in hard cohesive or dense granular material and 20 feet in soft cohesive or loose granular material and that piles for trestle bents penetrate a minimum of one-third of the unsupported length [AASHTO-LRFD 2$^{nd}$ Edition 10.7.1.2]. These minimums do not appear in the 3$^{rd}$ or 4$^{th}$ Editions. They also do not account for minimum penetration required for a point of fixity with p-y analysis. The minimum penetration for fixity was determined in the general study and is given in the P10L table.

- **Frame moments:** Because of the partial or full fixity at the pile head to superstructure connection, a steel H-pile will be subjected to moment in the plane parallel with centerline of roadway. If the bridge has no skew the moment will be about one axis of the pile, but if the bridge is skewed the moment will be about both axes because the piles will be skewed with webs parallel with the pile bent cap. Generally the moment from the superstructure (which will be maximum at the top of the pile) is considerably larger than the moment due to temperature movement (which will be approximately equal at the top and bottom of the pile) or water loads. Therefore, the pile head moment(s) will be the controlling moment condition.

  The amount of moment applied to the pile depends on the relative stiffnesses of the pile and superstructure. Generally the pile is more flexible than the superstructure and thus is subjected to relatively small moments. However, if the height of the pile above ground is relatively small, the pile will be stiffer and will attract more moment. For the general study, the minimum ground to cap height was set at 7 feet to check the stiffer pile condition.

  Moments from the superstructure need to be determined from a frame analysis, but moments from temperature movement and water loads can be estimated accurately from standard formulas.

- **Analysis observations:** A simple two-dimensional strip frame analysis in STAAD appears to be conservative for a skewed structure analyzed by grid analysis in STAAD.

  An LPILE analysis will be different from a Davisson analysis [AASHTO-LRFD 10.7.3.13.4]. For typical cases LPILE will show a similar depth to fixity but larger buckling lengths.

- **Design and analysis assumptions in other states:** Alabama researchers [Hughes et al. 2007] who investigated pile bent buckling assumed jointed superstructures but longitudinal lateral support would exist if small amounts of sway caused joints to close. One-foot pile embedment in the cap was considered as 50% fixity (which is appropriate based on earlier research [Castilla et al. 1984]). With 50% fixity at the cap and the pile fixed below ground, K could be estimated as 0.577, but the researchers considered this case to be unconservative. Except for very poor soil conditions, a subgrade modulus less than 5 lb/in$^3$, HP 10x42 piles were assumed to be fixed 5 feet below ground. An alternate assumption of 50% fixity at the cap and 50% fixity at the ground line (with the reduced column length) leads to a K of 0.707.

  In the past North Carolina used stringent design criteria for design of pile bents: K = 2.1 and 1-inch maximum lateral deflection under lateral load [Robinson et al. 2006]. These criteria were necessary because pile bents were used with jointed superstructures; the sections of the superstructure could not be expected to provide lateral bracing or to transfer horizontal loads to abutments.

  The Illinois design manual [IDOT 2006] indicates a one-foot pile embedment. In the longitudinal direction piles are assumed to be cantilevers. In the transverse direction piles may be modeled as cantilevers or as a pile-cap frame

- **Design procedures from other states: (Determining preliminary configuration of the pile bent and the magnitudes of loads is common to all procedures.)**

  North Carolina:
  (1) Evaluate soil conditions.
  (2) Run LPILE to determine point of fixity at maximum negative moment or maximum negative deflection.
  (3) Analyze structure as a frame with piles fixed at point of fixity.
  (4) Design piles [Possiel 2008].

  Louisiana Simplified LRFD for small, non-critical projects:
  (1) Determine point of fixity as 5 feet below ground or scour line (Scour is a minimum of 5 feet.).
  (2) Check slenderness limit, $L/d \leq 20$
  (3) Use tabulated axial compressive resistance to design piles [Bridge Design Section 2008].

  This method was checked for an expansion bent with various LRFD load combinations and found to be adequate for wind loads not exceeding 55 mph [Ferdous 2007]. For H-piles the method permits the following.

**Louisiana simplified LRFD method**

| Pile shape | Maximum factored live plus dead load, $P_u$, kips | Maximum height (H), feet |
|---|---|---|
| HP 10x42 | 170 – 230 | 11.67 |
| HP 10x57 | 230 – 300 | 11.67 |
| HP 14x73 | 290 – 380 | 18.33 |
| HP 14x89 | 360 - 470 | 18.33 |

(This simplified method permits greater load but less height than the P10A sheet.)

Louisiana Detailed LRFD:

(1) Determine point of fixity using Davisson method [AASHTO LRFD 10.7.3.13.4].
(2) Model superstructure in STAAD, and determine pile loads.
(3) Use LPILE to determine point of fixity based on loads.
(4) Revise STAAD model for second point of fixity.
(5) Design piles [Bridge Design Section 2008].

**Analysis and design criteria for the revised general study of pile bents, on which the Bridge Design Manual simplified method is based**

**Pile head**

- Monolithic cap, 12-inch pile embedment
  - Assume partial fixity at bottom of cap for x-axis and y-axis.
  - Apply 50% of moments from STAAD strip frame for $M_{ux}$ and $M_{uy}$.
  - Apply strip frame factored axial loads, which usually will be less than full factored pile axial load, $P_u$ (A*6*1.45).
- Non-monolithic cap, 18-inch pile embedment
  - Assume full fixity at bottom of cap for x-axis.
  - Assume partial fixity at top of cap for y-axis.
  - Apply full moment from STAAD strip frame for $M_{ux}$.
  - Apply 50% of moment from STAAD strip frame for $M_{uy}$.
  - Apply strip frame axial factored loads, which usually will be less than full factored pile axial load, $P_u$.

**Pile toe**

- Determine location of fixity by Davisson method neglecting encasement.
  - Consider y-axis only.
  - Use average depth to fixity for loose sand ($N \approx 5$) and soft clay ($N \approx 3$) and round to nearest foot. (For HP-10, use 6 feet; for HP-12, use 7 feet; and for HP-14, use 8 feet.)

**Buckling length (KL)**

- Use LPILE for soft clay (c = 2.6 psi, unit weight = 0.0666 pci, $E_{50}$ = 0.02).
- For fixed head pile, take KL = 2 times the distance from pile head to first inflection point or the distance between the first two inflection points, whichever controls.
- For pinned head pile, take KL = distance from pile head to first inflection point. LPILE generally indicates a decreasing KL as the axial load increases to the buckling load. For checking slenderness and pile resistance at the strength limit state, use the KL at the buckling load.
- For partially fixed head pile, take KL = average of fixed head and pinned head KLs.

**Strip frame analysis, factored loads with STAAD PDELTA analysis**

- Modeling
  - Locate the superstructure strip member at the centroid of the superstructure. Use a concrete member to model the depth of the superstructure at each abutment and pile bent cap.

    - Assume the superstructure strip is supported at each abutment by one HP of most appropriate stiffness for the strip width, oriented for weak axis bending in a prebored hole. (Usually this will be an HP 10x42.)
- J-series
    - Take strip width as distance between pile bent piles, perpendicular to centerline of roadway.
    - Take live load and dynamic load allowance spread over a 10-foot width. Proportion that load for the strip width based on pile spacing.
    - Neglect any load increase at wheel lines, and neglect any multiple presence factor.
- H-series
    - Take strip width as distance between pile bent piles, perpendicular to centerline of roadway.
    - Take live load and dynamic load allowance spread over a 10-foot width. Proportion that load for the strip width based on pile spacing.
    - Proportion stiffness of composite beam and deck to the strip width based on pile spacing.
    - Neglect any load increase for a beam bearing directly over a pile, and neglect any multiple presence factor.

**Strength I loads (Other strength limit states will not control.)**

- Consider DC, DW, LL, and IM with usual load factors.
- Consider TU with load factor of one for H-piles and pipe piles, which is equivalent to a concrete superstructure with 50-degree expansion or contraction. This is according to the text of AASHTO LRFD 3.4.1, p3-11 for steel substructures.
- Neglect WA load that will give a small moment for the x-axis at the pile head, but indirectly consider the load with $C_m$ for moment magnification. (Assume use of full encasement for deep streams or streams with high velocity.)
- Neglect BR load. Assume the jointless superstructure transmits BR loads directly to abutments.

**Design checks**

- Check KL/r < 120 for main members.
- For beam-column checks use braced frame moment magnification. Use $C_m$ for lateral loads between brace points permissible under the 2005 AISC Specification (assuming some small WA uniform load on pile). This $C_m$ reduction is not permitted by the AASHTO-LRFD Specifications [4.5.3.2.2b]. Using the AISC $C_m$ is a compromise between completely neglecting any lateral load, in which case there would be very little, if any, moment magnification, and taking $C_m = 1.0$.
- Check critical $P_u$, $M_{ux}$, and $M_{uy}$ beam-column combinations from strip analysis.
- For comparison only, apply full factored pile axial load, $P_u$ (6 ksi on area with 1.45 load factor) with largest strip frame moments.
- For comparison only, apply full factored load, $P_u$, with 2-inch eccentricity about each axis and using $K_x = K_y = 0.8$. (This will be similar to John Harkin's P10A pile analysis.)

**Revised general study of J-series and H-series pile bent piles**

With the criteria above, the following table was developed with proposed pile heights. For comparison the table also shows the pile heights from the P10A sheet and the initial P10L sheet.

| H-pile | Bearing, tons ($P_u$, kips) | H-dimension, feet | | Proposed H-dimension, feet | |
|---|---|---|---|---|---|
| | | P10A | P10L | Monolithic cap [(1)] | Non-monolithic cap [(1)] |
| HP 10x42 | 37 (107.88) | 16 | 13 | 19 | 15 |
| HP 10x57 | 50 (146.16) | 16 | 14 | 19 | 16 [(2)] [1.06+] |
| HP 12x 53 | 46 (133.85) | 26 | 18 | 23 | 20 [(2)] [1.05+] |
| HP 14x73 | 64 (186.18) | --- | 25 | 28 [(3)] | 25 [(2, 3)] [1.00+] |
| HP 14x89 | 78 (227.07) | --- | 26 | 29 [(3)] | 26 [(2, 3)] [1.02+] |

Table notes:

(1) Much of the difference between the H-values for the different caps is due to the use of a soft joint above the 3-foot deep non-monolithic cap.

(2) These H-dimensions will exceed a beam-column check by up to 7% as indicated by the combined force and moment ratio in brackets if the full $P_u$ is used with the maximum factored moments from a strip analysis for a maximum length bridge. This condition did not occur in the strip analyses investigated and is for comparison only.

(3) HP-14 piles are not proposed for greater heights because of the $K_y$ determined specifically for each pile in the revised general study. For the general study the $K_y$ was determined from an average of several pile shapes.

**Comments**

- For the new (2009) J24-series bridges the required bearing (with dynamic load allowance) varies from 32 to 42 tons ($P_u$ = 92.8 to 121.8 kips). Pile bent loads for the new J24-series will require load capacity of HP 10x42 to 130-foot bridge length and HP 12x53 to 150-foot length. Larger piles may be required based on height.

- For the old (2006) H24-series the required bearing (with dynamic load allowance) varies from 40 to 50 tons ($P_u$ = 116.00 to 145.00 kips). Pile bent loads for the old H24-series will require HP 12x53 to HP 10x57 depending on bridge length and number of piles in the bent. Larger piles may be required based on height.

**Bridge Design Manual table**

**Permissible heights, nominal resistances, and maximum plan sheet bearing for Grade 50 H-piles in pile bents**

| H-pile section | Maximum height, H (1), feet | | Recommended minimum height, H (2), feet | Minimum soil penetration (3) feet | Nominal Resistance, $P_n$ kips (4) | Max. plan sheet bearing, ton |
|---|---|---|---|---|---|---|
| | Monolithic cap | Non-monolithic cap | | | | |
| HP 10x42 | 19 | 15 | 7 | 18 | 154 | 37 |
| HP 10x57 | 19 | 16 | 7 | 18 | 208 | 50 |
| HP 12x53 | 23 | 20 | 9 | 21 | 192 | 46 |
| HP 14x73 | 28 | 25 | 13 | 24 | 265 | 64 |
| HP 14x89 | 29 | 26 | 13 | 24 | 324 | 78 |

Table notes:

(1) H is measured from bottom of cap.

(2) As the pile becomes shorter it becomes stiffer and attracts more moment but has greater axial capacity. Consult with the supervising Unit Leader before using a lesser height.

(3) These values ensure that a pile is sufficiently long so as to have a point of fixity. They were determined for the Davisson method [AISC 1986] and then checked with LPILE runs [Wang and Reese 1993]. In very few cases will LPILE require a greater penetration. If penetration is less than the table minimum it may be subject to “fence-posting”, and the pile requires special analysis.

(4) These nominal resistances are intended to be used with $\varphi_c = 0.70$. These values at Structural Resistance Level - 1 (SRL-1) were calculated from 1.45*6 ksi*A/0.7, which simplifies to 12.43*A.

Construction

J-series bridges are constructed with falsework and, therefore, construction loading of pile bent piles will be minimal. For H-series bridges, however, construction loading may be significant.

During the general study two construction cases for an HP 10x57 in an H-series bridge were checked: (1) cap pour and (2) C-beam placement. Reduced K-values considering the encasement were used at both stages but, even with the reduced K-values, the pile did not meet the AASHTO $KL/r \leq 120$ limit for primary members. For the construction stage the limit presumably does not apply.

At the cap pour stage, a maximum permissible height HP 10x57 at maximum H40 pile spacing, with no bracing, and with one-inch eccentricities and moment magnification, resulted in a combined axial compression and flexure ratio less than 0.400. The ratio is low enough that other pile shapes for H-series bridges also should be adequate.

For the beam placement, the condition was taken as an H-series 243-foot bridge, C-beam placement at 0-degree skew to maximize y-axis moment, maximum permissible height HP 10x57, maximum pile spacing, and no y-axis bracing. The combined axial compression and flexure ratio was less than 0.600 with reduced $K_y$ for encasement and, for comparison only, less than 1.100 with $K_y = 2.1$. The ratio with encasement is low enough that other pile shapes also should be adequate.

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## Pile bents with prestressed concrete piles ~ 17 December 2008

**Contents**



**Summary**

For J-06-series, H-06-series, and similar bridges the present allowable P10A loads for 14-inch and 16-inch prestressed concrete piles, 33 tons and 38 tons, may be extrapolated to the strength level using the average load factor 1.45 and compression phi factor 0.75. The nominal axial resistance for a 14-inch pile thus would be 127 kips, and the nominal resistance for a 16-inch pile would be 146 kips.

The respective maximum heights (H) would be 18 feet and 22 feet, an increase in height for the 14-inch pile. Generally it would be advisable not to use heights (H) less than 7 feet, especially if soils near the surface are relatively stiff.

**Permissible heights (H), nominal resistances, and maximum plan sheet bearings for prestressed concrete piles in pile bents**

| Square pile size, inches | Maximum height, H, feet | Recommended minimum height, H, feet | Minimum soil penetration feet (2) | Nominal Resistance, $P_n$ kips | Max. plan sheet bearing, tons |
|---|---|---|---|---|---|
| 14 | 18 | 7 | 24 | 127 | 33 |
| 16 | 22 | 7 | 27 | 146 | 38 |

Table notes:

(1) $\varphi_c = 0.75$

(2) These values ensure that pile is sufficiently long so as to have a point of fixity. They were determined for the Davisson method [AISC 1986] but then reduced based on LPILE runs [Wang and Reese 1993]. In very few cases will LPILE require a greater penetration. If penetration is less than the table minimum, pile requires special analysis.

These nominal resistances are sufficient for all H-06-series bridges but not for all J-06-series bridges.

Since the P10A sheet was developed AASHTO has added to the minimum pile spacing rule. The original minimum spacing of 2.5 feet now only is applicable for piles 12 inches or less in size. With the additional 2.5D rule, for a 14-inch pile the minimum spacing is 2.92 feet, and for a 16-inch pile is 3.33 feet. In some cases the J-06-series and H-06-series pile spacings are less and violate the AASHTO rule.

The AASHTO LRFD Specifications no longer specify minimum pile penetration: 10 feet in hard cohesive or dense granular material, 20 feet in soft cohesive or loose granular material, or one-third of the unsupported height (H) of a pile in a pile bent. In most cases, however, the minimum penetration in the table above will control, unless the designer specially analyzes the pile.

**History**

The first P10A sheet was issued in 1959 and had five trestle pile types (plus four foundation pile types) including the three on the present P10A sheet. Bearing values and maximum height (H) dimensions on the present sheet are the same as they were on the 1959 sheet. For the prestressed concrete pile type the initial prestress on the present sheet has been increased slightly from the 1959 sheet, from 168 kips to 174 kips for the 14-inch pile and from 227 kips to 231 kips for the 16-inch pile.

Although no office records exist with respect to the loads and heights permitted for the prestressed concrete piles on the P10A sheet, the allowable loads seem to follow the pattern of side dimensions: 33/38 = 0.87 and 14/16 = 0.88. The allowable heights (with an assumed depth to fixity of 6 feet based on past office practice), however, do not follow a simple L/D ratio: (13+6)/1.1667 = 16.3 for the 14-inch pile and (22+6)/1.3333 = 21.0 for the 16-inch pile. The ratio for the 16-inch pile is considerably less conservative than the ratio for the 14-inch pile.

Prestressed concrete pile computations, evidently by someone in the office and with a memo dated 4 December 1990, indicate that the allowable load equation for axial compression was recommended for $h'/r \leq 60$ [PCI 1993]. Using the h'/r limit, the maximum unbraced lengths for use of the equation would be 20.2 feet for 14-inch piles and 23.1 feet for 16-inch piles. Considering a pile braced and hinged at the top and fixed at 6 feet below ground (for "scour and mud" as assumed in the computations), the K could be taken as 0.8, and the permissible "H" for the equation would be 19.2 feet and 22.9 feet for the 14-inch and 16-inch piles, respectively. These "H" values compare with 13 feet and 22 feet on the P10A sheet.

In the office pile computations, minimum eccentricity was taken as (0.6 + 0.03h), a value for compression members in the ACI Code, which gives 1.02 inches for 14-inch piles and 1.08 inches for 16-inch piles. This eccentricity is greater than the recommended minimum value for piles in the 6th Edition of the *PCI Design Handbook* [PCI 2004], e = 0.05d, which is 0.7 inches for 14-inch piles and 0.8 inches for 16-inch piles.

Overall, the computations showed that the piles were more than adequate for moments caused by minimum eccentricity at the 33-ton and 38-ton allowable loads shown on the P10A sheet for 14-inch and 16-inch piles, respectively. The computations had no indication of how the depth to fixity and how the assumption of pinned head were determined.

**Issues**

- **Loading and pile head bracing:** See "Pile bents with steel H-piles."

- **Fixity at tops of piles embedded in monolithic or non-monolithic caps:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require an embedment of 12.0 minimum or a reduced embedment of 6.0 minimum with embedded dowel, bar, or strand attachment. The P10A sheet requires 12-inch embedment with dowels for piles in monolithic caps (J-06-series bridges) and 18-inch embedment without dowels for non-monolithic caps (J-06- or H-06-series bridges). The P10A sheet requirements exceed the AASHTO minimums.

  Various studies of prestressed concrete piles have indicated that a pile head with only 3-inch embedment and some reinforcing will perform as a partially fixed connection, not as a pinned connection as had been assumed by some states [Rollins and Stenlund 2008]. A greater embedment will cause greater fixity [Mokwa and Duncan 2003], and embedment equal to the pile size, with a 12-inch minimum, can result in a connection equal to the strength of the pile [Harries and Petrou 2001]. An earlier PCI document [PCI 1993] stated: "A pile fixed to a cap must be adequately reinforced with dowels…in order to resist bending moment."

  Based on the various tests and studies conducted to date, the P10A connections for prestressed concrete piles may be classified as partially fixed at the monolithic J-06-series cap and fully fixed to the J-06-series and H-06-series non-monolithic cap.

- **Fixity at non-monolithic cap to superstructure connection:** Possiel conducted tests of a prestressed beam to bent cap connection made with two elastomeric pads (similar to the H-06-series connection, except without the keyed-in diaphragm) and found that the connection resulted in partial fixity [Possiel 2008].

  Based on this testing and analysis, the J-06- and H-06-series non-monolithic cap to superstructure connection, out-of-plane, may be classified as partially fixed. In the plane of the pile bent, where the full width of the superstructure is engaged, the connection should be considered fixed.

- **Fixity at bottoms of piles:** The AASHTO LRFD Specifications give an approximate method to determine point of pile fixity [AASHTO-LRFD 10.7.3.13.4] developed in the 1960s by Davisson. At this time the method is known to be unconservative to some [Wilson et al. 2005] but not to all [Kumar et al. 2007]. Although the method was permitted for final design in the 2$^{nd}$ Edition of the AASHTO LRFD Specifications, in the 3$^{rd}$ and 4$^{th}$ Editions the method is to be used for preliminary design only. A more accurate point of fixity based on p-y analysis was developed for integral abutment piles [Greimann et al. 1987], and it was shown to vary for lateral load, moment, and buckling. The most recent method to estimate point of fixity and analyze driven piles is to use LPILE or comparable software that utilizes the p-y method.

  With LPILE (previously COM624P), determining the point of fixity seems to be open to engineering judgment. In an integral abutment example, engineers at the Tennessee DOT took the location of first zero deflection below ground as the fixed point [Wasserman and Walker 1996], whereas for pile bents, engineers at the North Carolina DOT take the fixed point as either the maximum negative deflection or the maximum negative moment below ground [Possiel 2008]. The assumptions associated with negative deflection or negative moment seem to have more validity.

  The point of fixity will vary depending on the stiffness and pinned, partially fixed, or fully fixed head condition of a pile [Possiel 2008]. The location will be deeper for a stiffer pile. It usually will be deeper for a fully fixed pile head, but the depth will depend on whether it is determined as first zero deflection, maximum negative deflection, or maximum negative moment.

  For the pile to have a point of fixity the pile must penetrate into the soil sufficiently so that there are at least two points of zero deflection below ground [Wang and Reese 1993]. If the pile does not penetrate far enough the bottom of the pile will deflect laterally in a “fence-posting” mode of behavior.

- **Buckling length and K-value:** With LPILE the buckling length can be determined as the distance between points of zero moment [Wasserman and Walker 1996]. For a pinned-head pile this distance is from the pile head to the first point of zero moment on the pile below. For a fixed-head pile, one distance is between the first two points of zero moment on the pile [Wasserman and Walker 1996], but there also is a distance above the first point that should be doubled to make a full buckling curve. This upper distance seems to be recognized but incorrectly analyzed in the literature [Wang and Reese 1993] and is difficult to determine if the pile head is not held to a fully fixed head zero slope, which gives a warning in LPILE. When using the buckling lengths determined from LPILE, the K-value should be taken as 1.0 because the lengths are for the full buckling curve.

  The AASHTO LRFD Specifications limit the K value to 1.0 for concrete members braced against sidesway unless analysis shows that a lower value may be used [AASHTO-LRFD 5.7.4.3].

  After comparing buckling lengths determined using the Davisson method for point of fixity and LPILE, it appears that establishing fixity with the Davisson method and using K = 1.0 usually will approximate the results from LPILE. Because of the simplicity of the Davisson method it gives more consistent results than LPILE and, for a general study, the Davisson method with K = 1.0 is more convenient.

- **Minimum pile spacing:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require that piles be spaced center-to-center not less than 30.0 inches or 2.5 pile diameters, whichever is larger. (The two-part rule has been in the AASHTO Standard Specifications since the late 1980s or early 1990s.) Based on width, for 14-inch piles the minimum spacing is 35 inches or 2.92 feet, and for 16-inch piles is 40 inches or

3.33 feet. For J24 series bridges 120 feet or more in length and skewed 30 degrees or less, pile bent piles are spaced too close to meet the minimum spacing requirements for one or both prestressed pile sizes. For H40 series bridges piles are spaced too close for some of the bridges 163.83 feet long and greater, depending on skew.

- **Eccentricity:** PCI recommends that piles loaded in compression be designed for a minimum eccentricity of 0.05 times the pile diameter or width [PCI 2004]. For 14-inch and 16-inch piles the minimum eccentricities are 0.7 inches and 0.8 inches, respectively. An earlier PCI document [PCI 1993] assumed an eccentricity of 0.1 times the pile diameter or width, without comment. The 1963 and 1971 ACI Codes required minimum eccentricities of 0.05 and 0.10 for spiral and tied columns, respectively, as a means to reduce maximum design strength. More recent ACI Codes use factors of 0.85 and 0.80 to limit design strength.

  The Iowa DOT Standard Specifications allow a pile head location tolerance of 4 inches for prestressed concrete piles [IDOT SS 2501.15]. Investigations of misplacement of H-piles have indicated that there is less effect of misplacement than expected because the very rigid cap distributes the effects of eccentricity to adjacent piles.

- **Battered piles:** Assume that increase in axial load in battered piles may be neglected. This assumption follows present office practice.

- **Scour:** Because pile bents are to be used only for sites with small streams and limited drainage area, some sites may be subjected to minor scour. In those cases the designer may simply consider the ground line to be at the maximum scour and choose a pile size for the increased height (H).

- **Pile penetration:** The 2nd Edition of the AASHTO LRFD Specifications required that piles penetrate a minimum of 10 feet in hard cohesive or dense granular material and 20 feet in soft cohesive or loose granular material and that piles for trestle bents penetrate a minimum of one-third of the unsupported length [AASHTO-LRFD 2nd Edition 10.7.1.2]. These minimums do not appear in the 3rd or 4th Editions. They also do not account for minimum penetration required for p-y analysis.

- **Frame moments:** Because of the partial or full fixity at the pile head to superstructure connection, prestressed concrete piles will be subjected to moment in the plane parallel with centerline of roadway. If the bridge has no skew the moment will be about one axis of the pile, but if the bridge is skewed the moment will be about both axes. Generally the moment from the superstructure is considerably larger than the moment due to temperature movement or water loads. Therefore, the pile head moment(s) will be the controlling moment condition.

  The amount of moment applied to the pile depends on the relative stiffnesses of the pile and superstructure. Generally the pile is more flexible than the superstructure and thus is subjected to relatively small moments. However, if the height of the pile above ground is relatively small, the pile will be stiffer and will attract more moment. For the general study, the minimum ground to cap height was set at 7 feet to check the stiffer pile condition.

  Moments from the superstructure need to be determined from a frame analysis, but moments from temperature movement and water loads can be estimated accurately from standard formulas.

- **Design procedures from other states: (Determining preliminary configuration of the pile bent and the magnitudes of loads is common to all procedures.)**

  North Carolina: (1) Evaluate soil conditions; (2) Run LPILE to determine point of fixity at maximum negative moment or maximum negative deflection; (3) Analyze structure as a frame with piles fixed at point of fixity; (4) Design piles [Possiel 2008].

  Louisiana Simplified LRFD for small, non-critical projects: (1) Determine point of fixity as 5 feet below ground or scour line (Scour is a minimum of 5 feet.); (2) Check slenderness limit, $L/d \leq 20$; (3) Use tabulated axial compressive resistance to design piles [Bridge Design Section 2008]. This method was checked for an expansion bent with various LRFD load combinations and found to be adequate for wind loads not exceeding 55 mph [Ferdous 2007]. If this method were used for a 14-inch prestressed concrete pile, the maximum height above ground would be 18.33 feet with a maximum factored dead plus live load of 110 to 170 kips. For a 16-inch pile, the maximum height would be 21.67 feet with a maximum factored dead plus live load of 140 to 200 kips. (This method is generally less conservative than the P10A sheet.)

Louisiana Detailed LRFD: (1) Determine point of fixity using Davisson method [AASHTO LRFD 10.7.3.13.4]; (2) Model superstructure in STAAD, and determine pile loads; (3) Use LPILE to determine point of fixity based on loads; (4) Revise STAAD model for second point of fixity; (5) Design piles [Bridge Design Section 2008].

**Analysis and design conditions for the general study of pile bents, on which the design manual simplified method is based, with notes for design of special bridges**

- Consider only the Strength I limit state. Strength III and V, which include wind, are not likely to control because the jointless superstructure acting as a wide horizontal beam will carry all except part of the substructure wind loads to the abutments. The superstructure also will carry braking forces to the abutments. Thus the loads in Strength I to consider are dead (DC and DW), live and dynamic load allowance (LL and IM), temperature (TU), and water (WA).
- A typical water load results in a very small moment at the pile head where the overall moment is maximum. For a general study, neglect the water load.
  Note: For checking a specific bridge, include the water load if applicable.
- Consider a pile placed at zero-degree skew and at 45-degree skew. These two cases are at the extremes of pile placement. At 45-degree skew, two-dimensional bridge frame analysis has shown the pile-head moments about each of the two axes to be $M_{z,\ zero\ skew}$*cos 45. Biaxial bending checks using the Bresler reciprocal load method are required [AASHTO-LRFD 5.7.4.5].
- For monolithic caps, the embedment is less than the pile size; therefore, consider the pile head to be 50% fixed. Analyze as if the pile head were fixed, and use 50% of the moment.
- For non-monolithic caps, the embedment is greater than the pile size; therefore, consider the pile head to be fixed in the plane of the bent. Out-of-plane, although the pile head would be fixed to the cap, the cap to superstructure connection is made with elastomeric pads and compressible material; therefore, consider the connection to be 50% fixed. Analyze as if the pile head were fixed, and use 50% of the moment.
- Take the location of fixity as computed by the approximate Davisson method for soft clay or loose sand [AASHTO-LRFD 10.7.3.13.4], and take K = 1.0 rather than a smaller value [AASHTO-LRFD 5.7.4.3]. This will approximate an LPILE analysis.
  Note: For checking or designing a specific bridge use LPILE with the bridge site soil data.
- In STAAD analyze a strip frame of tributary width for a pile bent pile in order to determine factored moments.
- For zero-degree skew, check the prestressed concrete pile with “PCI Prestressed Concrete Pile Interaction Diagram Spreadsheet,” considering factored axial load, single-axis factored moment, and slenderness.
- For 45-degree skew use the PCI spreadsheet and then the Bresler reciprocal load method [AASHTO-LRFD 5.7.4.5] to check biaxial bending.

**General study of J-06-series and H-06-series pile bent piles**

The table below summarizes two-dimensional STAAD frame runs for the following conditions:

- Loads are applied at Strength I magnitude.
- Live loads are not reduced for bridge width or a multiple presence factor. (This conservatism partially compensates for variations in pile loads due to spaced beam loads in the H-06-series bridges.)
- The superstructure frame strip is of a stiffness determined for pile bent pile spacing.
- At each abutment a single HP 10x42 abutment pile is fixed to the superstructure strip and fixed 15 feet below that strip. (Although the abutment pile spacing may not match the strip width, the stiffness of the pile is relatively small and will not significantly influence the pile bent pile, which is a span distance away from the abutment.)

- Pile bent piles are placed at a 45-degree skew. (A comparison of the checks for skewed and unskewed piles in the second table below indicated that the skewed pile would control.)
- At a monolithic cap, piles are fixed about each axis. (Partial fixity was assumed for design checks.)
- At a non-monolithic cap, piles are fixed in the plane of the bent but pinned out-of-plane. (Design checks assumed fixity and partial fixity, respectively.)
- 14-inch prestressed concrete piles are fixed 9 feet below ground, and 16-inch piles are fixed 10 feet below ground. (These distances are based on soft soil and the Davisson method. Although LPILE may indicate fixity at greater depths, the shallower depths will increase pile stiffness and result in conservative pile moments. The use of K = 1.0 approximates the LPILE buckling length.)

**Factored loads and moments from STAAD runs assuming fixed pile heads**

| Pile size, inches | STAAD run | Series, length, feet | Strip width, feet | H, feet | $P_u$, kips | $M_{ux}$ at top, in-k (bottom) | $M_{uy}$ at top, in-k (bottom) |
|---|---|---|---|---|---|---|---|
| 14 | 03FIX | J, 100 | 3.24 | 18 | 90.49 | 462.83 | 462.83 |
| | 03PIN | J, 100 | 3.24 | 18 | 92.33 | (27.40) | (27.40) |
| | 03FXS | J, 100 | 3.24 | 7 | 93.28 | 746.32 | 746.32 |
| | 07FXP | J, 100 | 3.24 | 18 | 103.40 | 457.58 | (22.18) |
| | 10FXP | H, 243 | 1.77 | 18 | 84.80 | 312.33 | (46.15) |
| 16 | 04FIX | J, 140 | 2.47 | 22 | 108.11 | 732.48 | 732.48 |
| | 04PIN | J, 140 | 2.47 | 22 | 109.45 | (46.90) | (46.90) |
| | 04FXS | J, 140 | 2.47 | 7 | 102.09 | 1262.96 | 1262.96 |
| | 08FXP | J, 140 | 2.47 | 22 | 113.71 | 752.99 | (39.17) |
| | 11FXP | H, 243 | 2.15 | 22 | 103.13 | 431.61 | (57.87) |

Table notes

(1) Using an average load factor of 1.45, maximum P10A loads at factored strength limit state are 95.7 kips and 110.2 kips for 14-inch and 16-inch piles, respectively. The loads determined from the strip analysis approximately check the J-06- and H-06-series loads.

(2) Moments for short piles are larger than moments for maximum height piles, which are more flexible.

(3) Moments are larger for J-06-series bridges than for comparable H-06-series bridges.

(4) Controlling cases to be checked are indicated with yellow shading.

The table below summarizes the controlling checks with “PCI Prestressed Concrete Pile Interaction Diagram Spreadsheet” assuming the proposed partial or full fixity at pile heads. The spreadsheet includes the effects of slenderness.

**Checks of controlling cases using PCI Spreadsheet**

| STAAD run, pile size, in | Skew, H, feet | $P_u$, kips | $M_{ux}$, ft-k | % fixity | $M_{uy}$, ft-k | % fixity | $\varphi P_n$, kips permissible |
|---|---|---|---|---|---|---|---|
| 03FXS, 14 | 0, 7 | 93.28 | 44 | 50 | --- | --- | 405 |
| | 45, 7 | 93.28 | 31 | 50 | 31 | 50 | 354 |
| 07FXP, 14 | 0, 18 | 103.40 | 27 | 50 | --- | --- | 258 |
| | 45, 18 | 103.40 | 38 | 100 | 19 | 50 | 158 |
| 04FXS, 16 | 0, 7 | 102.09 | 74 | 50 | --- | --- | 510 |
| | 45, 7 | 102.09 | 53 | 50 | 53 | 50 | 449 |
| 08FXP, 16 | 0, 22 | 113.71 | 44 | 50 | --- | --- | 310 |
| | 45, 22 | 113.71 | 63 | 100 | 31 | 50 | 144 |

Table notes

(1) The controlling $P_u$ values would correlate with nominal resistances of 137 and 151 kips for 14-inch and 16-inch piles, respectively.

(2) The taller piles at 45-degree skew control, as indicated by the yellow shading. At the controlling conditions there is not much overcapacity.

(3) The equivalent eccentricities for the controlling cases are 4.4 inches and 2.2 inches for the x and y axes of the 14-inch pile and 6.6 inches and 3.3 inches for the x and y axes of the 16-inch pile. These are well above minimum eccentricities

(4) By computations not shown in the table, assumed full fixity for the 16-inch pile caused some checks to fail.

Under construction conditions two cases for the 14-inch pile were checked: (1) cap pour and (2) C-beam placement. For the cap pour the pile had extra capacity, but for the beam placement the pile had virtually no extra capacity.

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## Pile bents with concrete-filled steel pipe piles ~ 17 December 2008

**Contents**



**Summary**

For J-06-series, H-06-series, and similar bridges the present allowable P10A loads for 14-inch and 16-inch steel pipe piles, 30 tons and 36 tons, may be increased to the same values as for prestressed concrete piles, 33 and 38 tons. If these bearing values are extrapolated to the strength level using the average load factor 1.45 and compression phi factor 0.80, the nominal axial resistance for a 14-inch pile thus is 119 kips, and the nominal resistance for a 16-inch pile is 137 kips.

The respective maximum heights (H) based on the general study are the same as for prestressed concrete piles, 18 feet and 22 feet, an increase in height for the 14-inch pile. Generally it would be advisable not to use heights (H) less than 7 feet, especially if soils near the surface are relatively stiff.

**Permissible heights (H), nominal resistances, and maximum plan sheet bearings for concrete-filled steel pipe piles in pile bents**

| Pile diameter inches (1) | Maximum height, H, feet | Recommended minimum height, H, feet | Minimum soil penetration feet (2) | Nominal Resistance, $P_n$ kips (3) | Max. plan sheet bearing, tons |
|---|---|---|---|---|---|
| 14 | 18 | 7 | 24 | 119 | 33 |
| 16 | 22 | 7 | 27 | 137 | 38 |

Table notes:

(1) Minimum wall thicknesses are: 14-inch: Grade 2-0.219 inch, Grade 3-0.203 inch; 16-inch: Grade 2-0.230 inch, Grade 3-0.230 inch.

(2) These values ensure that the pile is sufficiently long so as to have a point of fixity. They were determined for the Davisson method [AISC 1986] but then reduced based on LPILE runs [Wang and Reese 1993]. In very few cases will LPILE require a greater penetration. If penetration is less than the table minimum, the pile requires special analysis.

(3) $\varphi_c = 0.80$

These nominal resistances are sufficient for all H-06-series bridges but not for all J-06-series bridges.

Since the P10A sheet was developed AASHTO has added to the minimum pile spacing rule. The original minimum spacing of 2.5 feet now only is applicable for piles 12 inches or less in size. With the additional 2.5D rule, for a 14-inch pile the minimum spacing is 2.92 feet, and for a 16-inch pile is 3.33 feet. In some cases the J-06-series and H-06-series pile spacings are less and violate the AASHTO rule.

The AASHTO LRFD Specifications no longer specify minimum pile penetration: 10 feet in hard cohesive or dense granular material, 20 feet in soft cohesive or loose granular material, or one-third of the unsupported height (H) of a pile in a pile bent. In most cases, however, the minimum penetration in the table above will control, unless the designer specially analyzes the pile.

**History**

The first P10A sheet was issued in 1959 and had five trestle pile types (plus four foundation pile types) including the three on the present P10A sheet. Bearing values and maximum height (H) dimensions on the present sheet are the same as they were on the 1959 sheet. For the pipe pile type the weights given on the P10A sheet no longer match the weights in the ASTM standard [A 252 – 98 (Reapproved 2007)].

Although no office records exist with respect to the loads and heights permitted for the pipe piles on the P10A sheet, the allowable loads seem to approximate the pattern of side dimensions: 30/36 = 0.83 and 14/16 = 0.88. The allowable heights (with an assumed depth to fixity of 6 feet for “scour and mud” based on past office practice), however, do not follow a simple L/D ratio: (13+6)/1.1667 = 16.3 for the 14-inch pile and (22+6)/1.3333 = 21.0 for the 16-inch pile. The ratio for the 16-inch pile is considerably less conservative than the ratio for the 14-inch pile.

**Issues**

- **Loading and pile head bracing:** See “Pile bents with steel H-piles.”

- **Pipe wall thickness:** Pipe wall thickness is not given on the P10A sheet, but the thickness can be determined from the listed weight as indicated in the table below. Note that the P10A sheet weights do not match the present ASTM standard [A252-98 (2007)], and the thicknesses for Grade 3 pipe do not meet the AASHTO minimum [AASHTO-LRFD 6.9.4.2].

**Weights and wall thicknesses of P10A pipe piles**

| Pipe diameter, inches | Grade 2 (35 ksi) | | Grade 3 (45 ksi) | |
|---|---|---|---|---|
| | Weight, lb/ft P10A/ASTM | Thickness, inches P10A/AASHTO min [6.9.4.2] (1) | Weight, lb/ft P10A/ASTM | Thickness, inches P10A/AASHTO min [6.9.4.2] (1) |
| 14 | 32.20/32.26 | 0.219/0.179 | 27.66/27.76 | 0.188/0.203 |
| 16 | 38.74/38.77 | 0.230/0.203 | 36.87/36.95 | 0.219/0.230 |

Table notes:

(1) These thicknesses are the minimum available pipe thicknesses larger than the computed AASHTO minimum. The referenced AASHTO article does permit the designer to substitute a lower, computed maximum compressive stress due to the factored axial force and bending moment, and the computed stress can reduce the minimum thickness requirement.

(2) Shaded table cells indicate differences between the P10A sheet and standards.

The following are additional comments on pipe pile wall thickness:

- The thickness in the table does not allow for corrosion.
- Nebraska DOR uses a 0.188-inch minimum thickness.
- Missouri DOT uses a 0.25-inch minimum for 14-inch piles and a 0.375-inch minimum for 16-inch piles to avoid damage during construction.
- One author has recommended a general minimum pipe pile size of 10 inches and wall thickness of 5/16 inch [Teng 1962].
- The wall thicknesses in the table are less than the minimum thickness for structural steel, 5/16 inch, specified by AASHTO [AASHTO-LRFD 6.7.3], but it is unclear as to whether a concrete-filled pipe pile is considered structural steel. The Nebraska DOR and Missouri DOT obviously do not apply the rule.
- Lateral load testing discussed in a literature review [Stephens and McKittrick 2005] generally has been conducted on piles with 0.5-inch thickness, probably because of seismic considerations.

Considering the AASHTO rule for minimum thickness, as well as corrosion and potential for construction damage, it would be appropriate to increase the Grade 3 pile thicknesses to the AASHTO minimums in the table.

- **Comparisons with prestressed concrete piles**

  If the short-term composite action of the concrete fill in the steel pipe piles on the P10A sheet is considered, the stiffnesses (EIs) of the pipe piles very closely match the stiffnesses of the comparable concrete gross cross sections of the prestressed concrete piles. The table below shows the comparisons.

**Comparison of stiffnesses of P10A steel pipe piles and prestressed concrete piles**

| Steel pipe pile | | | | | Square prestressed concrete pile |
|---|---|---|---|---|---|
| Size, inches | Wall thickness, in | Grade | Composite I, $in^4$ | EI, $k\text{-}in^2$ | EI, $k\text{-}in^2$ |
| 14 | 0.219 | 2 | 410 | 11,890,000 | 12,903,001 |
| 14 | 0.188 | 3 | 383 | 11,107,000 | |
| 16 | 0.230 | 2 | 672 | 19,488,000 | 22,011,951 |
| 16 | 0.219 | 3 | 658 | 19,082,000 | |

Considering the relatively close stiffness match it will be reasonable to assume the same depth to fixity and same STAAD frame results for P10A steel pipe piles as for prestressed concrete piles. As a check, STAAD longitudinal strip frame results for 16 x 16-inch prestressed concrete piles and 16-inch diameter composite pipe piles were compared. The pipe pile results were slightly smaller as expected and, therefore, using the available prestressed pile results will be conservative.

- **Fixity at tops of piles embedded in monolithic or non-monolithic caps:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require an embedment of 12.0 minimum or a reduced embedment of 6.0 minimum with embedded dowel, bar, or strand attachment. The P10A sheet requires 12-inch embedment with dowels for piles in monolithic caps (J-06-series bridges) and 18-inch embedment without dowels for non-monolithic caps (J-06- or H-06-series bridges). The P10A sheet requirements exceed the AASHTO minimums.

  No tests were found in the literature that attempted to quantify the relative fixity of the pipe pile to cap connection based on pile embedment. However, testing by Montana researchers generally indicated an effective fixity for embedment equal to the diameter of the pile [Stephens and McKittrick 2005].

  Based on the various tests and studies conducted to date on other pile types, the P10A connections for steel pipe piles may be classified as partially fixed at the monolithic J-06-series cap and fully fixed to the J-06-series and H-06-series non-monolithic cap.

- **Fixity at non-monolithic cap to superstructure connection:** Possiel conducted tests of a prestressed beam to bent cap connection made with two elastomeric pads (similar to the H-06-series connection, except without the keyed-in diaphragm) and found that the connection resulted in partial fixity [Possiel 2008].

  Based on this testing and analysis, the J-06- and H-06-series non-monolithic cap to superstructure connection, out-of-plane, may be classified as partially fixed. In the plane of the pile bent, where the full width of the superstructure is engaged, the connection should be considered fixed.

- **Fixity at bottoms of piles:** The AASHTO LRFD Specifications give an approximate method to determine point of pile fixity [AASHTO-LRFD 10.7.3.13.4] developed in the 1960s by Davisson. At this time the method is known to be unconservative to some [Wilson et al. 2005] but not to all [Kumar et al. 2007]. Although the method was permitted for final design in the 2nd Edition of the AASHTO LRFD Specifications, in the 3rd and 4th Editions the method is be used for preliminary design only. A more accurate point of fixity based on p-y analysis was developed for integral abutment piles [Greimann et al. 1987], and it was shown to vary for lateral load, moment, and buckling. The most recent method to estimate point of fixity and analyze driven piles is to use LPILE or comparable software that utilizes the p-y method.

  With LPILE (previously COM624P), determining the point of fixity seems to be open to engineering judgment. In an integral abutment example, engineers at the Tennessee DOT took the location of first zero deflection below ground as the fixed point [Wasserman and Walker 1996], whereas for pile bents, engineers at the North Carolina DOT take the fixed point as either the maximum negative deflection or the maximum negative moment below ground [Possiel 2008]. The assumptions associated with negative deflection or negative moment seem to have more validity than the first zero deflection.

  The point of fixity will vary depending on the pile shaft stiffness and the amount of fixity at the pile head [Possiel 2008]. The point of fixity will be deeper for a stiffer pile. It will be deeper for a fully fixed pile head, but the depth will depend on whether it is determined as first zero deflection, maximum negative deflection, or maximum negative moment.

  For the pile to have a point of fixity the pile must penetrate into the soil sufficiently so that there are at least two points of zero deflection below ground [Wang and Reese 1993]. If the pile does not penetrate far enough the bottom of the pile will deflect laterally in a “fence-posting” mode of behavior.

- **Buckling length and K-value:** With LPILE the buckling length can be determined as the distance between points of zero moment [Wasserman and Walker 1996]. For a pinned-head pile this distance is from the pile head to the first point of zero moment on the pile below. For a fixed-head pile, one distance is between the first two points of zero moment on the pile [Wasserman and Walker 1996], but there also is a distance above the first point that should be doubled to make a full buckling curve. This upper distance seems to be recognized but incorrectly analyzed in the literature [Wang and Reese 1993] and is difficult to determine if the pile head is not held to a fully fixed head zero slope, which gives a warning in LPILE. When using the buckling lengths determined from LPILE, the K-value should be taken as 1.0 because the lengths are for the full buckling curve.

After comparing buckling lengths determined using the Davisson method for point of fixity and LPILE, it appears that establishing fixity with the Davisson method and using K = 1.0 (with the length from point of fixity to pile head) usually will approximate the results from LPILE. Because of the simplicity of the Davisson method it gives more consistent results than LPILE and, for a general study, the Davisson method with K = 1.0 is more convenient.

- **Minimum pile spacing:** The AASHTO LRFD Specifications [AASHTO LRFD 10.7.1.2] require that piles be spaced center-to-center not less than 30.0 inches or 2.5 pile diameters, whichever is larger. (The two-part rule has been in the AASHTO Standard Specifications since the late 1980s or early 1990s.) Based on width, for 14-inch piles the minimum spacing is 35 inches or 2.92 feet, and for 16-inch piles is 40 inches or 3.33 feet. For J24 series bridges 120 feet or more in length and skewed 30 degrees or less, pile bent piles are spaced too close to meet the minimum spacing requirements for one or both prestressed pile sizes. For H40 series bridges piles are spaced too close for some of the bridges 163.83 feet long and greater, depending on skew.

- **Eccentricity:** AISC has not established rules for minimum eccentricity of column axial load. It is common practice, however, to determine eccentricity of column load from connection details. The Iowa DOT Standard Specifications have no rules for misplacement of the heads of steel pipe piles (but allow up to 4 inches for prestressed piles, which also are relatively stiff displacement piles). Therefore, without tolerance rules for heads of pipe piles, any design eccentricity should be based on details or engineering judgment.

- **Battered piles:** Assume that increase in axial load in battered piles may be neglected. This assumption follows present office practice.

- **Scour:** Because pile bents are to be used only for sites with small streams and limited drainage area, some sites may be subjected to minor scour. In those cases the designer may simply consider the ground line to be at the maximum scour and choose a pile size for the increased height (H).

- **Pile penetration:** The 2$^{nd}$ Edition of the AASHTO LRFD Specifications required that piles penetrate a minimum of 10 feet in hard cohesive or dense granular material and 20 feet in soft cohesive or loose granular material and that piles for trestle bents penetrate a minimum of one-third of the unsupported length [AASHTO-LRFD 2$^{nd}$ Edition 10.7.1.2]. These minimums do not appear in the 3$^{rd}$ or 4$^{th}$ Editions. They also do not account for minimum penetration required for p-y analysis.

- **Frame moments:** Because of the partial or full fixity at the pile head to superstructure connection, steel pipe piles will be subjected to moment in the plane parallel with centerline of roadway. Generally the moment from the superstructure is considerably larger than the moment due to temperature movement or water loads. Therefore, the pile head moment(s) will be the controlling moment condition.

  The amount of moment applied to the pile depends on the relative stiffnesses of the pile and superstructure and the amount of head fixity. Generally the pile is more flexible than the superstructure and thus is subjected to relatively small moments. However, if the height of the pile above ground is relatively small, the pile will be stiffer and will attract more moment. For the general study, the minimum ground to cap height (H) was set at 7 feet to check the stiffer pile condition.

  Moments from the superstructure need to be determined from a frame analysis, but moments from temperature movement and water loads can be estimated accurately from standard formulas.

- **Design procedures from other states: (Determining preliminary configuration of the pile bent and the magnitudes of loads is common to all procedures.)**

North Carolina: (1) Evaluate soil conditions; (2) Run LPILE to determine point of fixity at maximum negative moment or maximum negative deflection; (3) Analyze structure as a frame with piles fixed at point of fixity; (4) Design piles [Possiel 2008].

Louisiana Simplified LRFD for small, non-critical projects: (1) Determine point of fixity as 5 feet below ground or scour line (Scour is a minimum of 5 feet.); (2) Check slenderness limit, $L/d \leq 20$; (3) Use tabulated axial compressive resistance to design piles [Bridge Design Section 2008]. This method was checked for an expansion bent with various LRFD load combinations and found to be adequate for wind loads not exceeding 55 mph [Ferdous 2007]. If this method were used for a 14-inch pipe pile without scour, the maximum height above ground would be 18.33 feet with a maximum factored dead plus live load of 170 kips. For a 16-inch pipe pile without scour, the maximum height would be 21.67 feet with a maximum factored dead plus live load of 260 kips. (This method is less conservative than the P10A sheet.)

Louisiana Detailed LRFD: (1) Determine point of fixity using Davisson method [AASHTO LRFD 10.7.3.13.4]; (2) Model superstructure in STAAD, and determine pile loads; (3) Use LPILE to determine point of fixity based on loads; (4) Revise STAAD model for second point of fixity; (5) Design piles [Bridge Design Section 2008].

**Analysis and design conditions for the general study of pile bents, on which the design manual simplified method is based, with notes for design of special bridges**

- Consider only the Strength I limit state. Strength III and V, which include wind, are not likely to control because the jointless superstructure acting as a wide horizontal beam will carry all except part of the substructure wind loads to the abutments. The superstructure also will carry braking forces to the abutments. Thus the loads in Strength I to consider are dead (DC and DW), live and dynamic load allowance (LL and IM), temperature (TU), and water (WA).
- A typical water load results in a very small moment at the pile head where the overall moment is maximum. For a general study, neglect the water load.
  Note: For checking a specific bridge, include the water load if applicable.
- Consider a pile placed in a zero-degree skew bridge and in a 45-degree skew bridge in order to identify maximum loading and controlling pile slenderness conditions. These two skews are the extremes for J-series and H-series bridges. Because of the circular, symmetrical shape of a pipe pile, in many cases the conditions about two perpendicular axes can be resolved to a single condition.
- For monolithic caps, the embedment is less than the pile size; therefore, consider the pile head to be 50% fixed. Analyze as if the pile head were fixed, and use 50% of the moment.
- For non-monolithic caps, the embedment is greater than the pile size; therefore, consider the pile head to be fixed in the plane of the bent. Out-of-plane, although the pile head would be fixed to the cap, the cap to superstructure connection is made with elastomeric pads and compressible material; therefore, consider the connection to be 50% fixed. Analyze as if the pile head were fixed, and use 50% of the moment.
- Take the location of fixity as computed by the approximate Davisson method for soft clay or loose sand [AASHTO-LRFD 10.7.3.13.4], and take K = 1.0 for the length from point of fixity to underside of monolithic cap or top of non-monolithic cap. This will approximate the buckling length from an LPILE analysis.
  Note: For checking or designing a specific bridge use LPILE with the bridge site soil data.
- In STAAD analyze a strip frame of tributary width for a pile bent pile in order to determine factored moments.
- Check the pile according to AASHTO LRFD specifications:
  - Pipe wall thickness [6.9.4.2]
  - Limiting slenderness ratio [6.9.3]
  - Nominal compressive resistance [6.9.5.1 and 6.9.5.2]
  - Flexure of concrete filled tubes [6.12.2.3.2]
  - Combined axial compression and flexure [6.9.2.2]

**General study of J-06-series and H-06-series pile bent piles**

The table below summarizes two-dimensional STAAD frame runs for the conditions listed below. Although the STAAD runs were for 14-inch and 16-inch square prestressed concrete piles, the relative stiffnesses of concrete filled 14-inch and 16-inch pipe piles are nearly the same, and the prestressed pile results can be used conservatively with the pipe piles. All items directly below, and the table apply to prestressed concrete piles.

- Loads are applied at Strength I magnitude.
- Live loads are not reduced for bridge width or a multiple presence factor. (This conservatism partially compensates for variations in pile loads due to spaced beam loads in the H-06-series bridges.)
- The superstructure frame strip is of a stiffness determined for pile bent pile spacing.
- At each abutment a single HP 10x42 abutment pile is fixed to the superstructure strip and fixed 15 feet below that strip. (Although the design abutment pile spacing may not match the strip width, the stiffness of the pile is relatively small and will not significantly influence the pile bent pile, which is a span distance away from the abutment.)
- The prestressed concrete piles are placed at a 45-degree skew. (A comparison of the checks for skewed and unskewed piles indicated that skewed piles would control.)
- At a monolithic cap, piles are fixed about each axis. (Partial fixity was assumed for design checks.)
- At a non-monolithic cap, piles are fixed in the plane of the bent but pinned out-of-plane. (Design checks assumed fixity and partial fixity, respectively.)
- 14-inch prestressed concrete piles are fixed 9 feet below ground, and 16-inch piles are fixed 10 feet below ground. (These distances are based on soft soil and the Davisson method. Although LPILE may indicate fixity at greater depths, the shallower depths will increase pile stiffness and result in conservative pile moments. The use of K = 1.0 with the length from point of fixity to pile head approximates the LPILE buckling length.)

**Factored loads and moments for checking pipe piles. (These loads and moments from STAAD runs for prestressed concrete piles will be nearly the same as for pipe piles because both pile types have nearly the same EI for the same size.)**

| Pile size, inches | STAAD run | Series, length, feet | Strip width, feet | H, feet | $P_u$, kips | $M_{ux}$ at top, in-k (bottom) | $M_{uy}$ at top, in-k (bottom) |
|---|---|---|---|---|---|---|---|
| 14 | 03FIX | J, 100 | 3.24 | 18 | 90.49 | 462.83 | 462.83 |
| | 03PIN | J, 100 | 3.24 | 18 | 92.33 | (27.40) | (27.40) |
| | 03FXS | J, 100 | 3.24 | 7 | 93.28 | 746.32 | 746.32 |
| | 07FXP | J, 100 | 3.24 | 18 | 103.40 | 457.58 | (22.18) |
| | 10FXP | H, 243 | 1.77 | 18 | 84.80 | 312.33 | (46.15) |
| 16 | 04FIX | J, 140 | 2.47 | 22 | 108.11 | 732.48 | 732.48 |
| | 04PIN | J, 140 | 2.47 | 22 | 109.45 | (46.90) | (46.90) |
| | 04FXS | J, 140 | 2.47 | 7 | 102.09 | 1262.96 | 1262.96 |
| | 08FXP | J, 140 | 2.47 | 22 | 113.71 | 752.99 | (39.17) |
| | 11FXP | H, 243 | 2.15 | 22 | 103.13 | 431.61 | (57.87) |

Table notes

(1) All bridges are skewed 45 degrees.
(2) Using an average load factor of 1.45, maximum P10A loads of 33 tons and 38 tons at the factored strength limit state are 95.7 kips and 110.2 kips for 14-inch and 16-inch prestressed concrete piles, respectively. The loads determined from the strip analysis approximately check the J-06- and H-06-series loads.
(3) Moments for short piles are larger than moments for maximum height piles, which are more flexible.
(4) Moments are larger for J-06-series bridges than for comparable H-06-series bridges.
(5) Controlling cases to be checked are indicated with yellow shading.

The table below summarizes the controlling checks with an Excel spreadsheet written to consider the various AASHTO rules associated with concrete-filled pipe piles.

**Checks of controlling cases for concrete-filled pipe piles**

| STAAD run | Pile dia., in; Grade | Skew; H, feet | $P_u$, kips | $M_{ux}$, in-k | $M_{uy}$, in-k | $M_{ur}$, in-k (2) | Check ratio (3) |
|---|---|---|---|---|---|---|---|
| 03FXS | 14; 2 | 45; 7 | 93.28 | 373.16 | 373.16 | 527.73 | 0.554 |
| | 14; 3 (1) | 45; 7 | 93.28 | 373.16 | 373.16 | 527.73 | 0.504 |
| 07FXP | 14; 2 | 45; 18 | 103.40 | 457.58 | 228.79 | 511.59 | 0.652 |
| | 14; 3 (1) | 45; 18 | 103.40 | 457.58 | 228.79 | 511.59 | 0.612 |
| 04FXS | 16; 2 | 45; 7 | 102.09 | 631.48 | 631.48 | 893.05 | 0.649 |
| | 16; 3 (1) | 45; 7 | 102.09 | 631.48 | 631.48 | 893.05 | 0.536 |
| 08FXP | 16; 2 | 45; 22 | 113.71 | 752.99 | 376.50 | 841.87 | 0.703 |
| | 16; 3 (1) | 45; 22 | 113.71 | 752.99 | 376.50 | 841.87 | 0.604 |

Table notes

(1) For the P10A Grade 3 piles the thicknesses do not meet the AASHTO local buckling requirement unless the computed compressive stress rather than the yield stress is considered [AASHTO-LRFD 6.9.4.2].

(2) $M_{ur}$ was determined by resolving $M_{ux}$ and $M_{uy}$, which were determined by considering pile axes in and out of the plane of the bent.

(3) The taller, Grade 2 piles control.

Under construction conditions two cases for the 14-inch pile were checked: (1) cap pour and (2) C-beam placement for an H40, 243-foot bridge. C-beam placement was the controlling condition, with an acceptable check of 0.418 < 1.000.

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