

A Pile Connection Primer

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Abstract

Analysis and design of pile to cap connections requires careful consideration of the loads and load transfer mechanisms involved. Connections are seldom covered in depth by Code Equations, and the engineer must apply a little ingenuity to develop a consistent and well founded design approach.

For design of pile to cap connection details it must be shown that there is sufficient strength at the critical section (usually the pile to cap interface), adequate anchorage in the cap above the pile to cap interface, as well as adequate anchorage in the pile below the pile to cap interface. To address each of these issues a different analysis is needed: The Ultimate Strength Interaction Diagram is needed for the capacity of the connection at the critical section(s), separate calculations are necessary to establish the strength & development of the cap connection, and additional calculations needed to establish the strength & development of the pile connection.

The authors will present several options for design of pile to cap connection details which have been used successfully, and which provide very economical solutions for design. These include flanged pipe sections, rebar or threadbar cages dropped in pile voids, and threadbar grouted in corrugated metal pipe. Calculations will be provided which are in accordance with AASHTO, PCI, and ACI Code Specifications.

Introduction

The purpose of this paper is to provide guidance for design of substructure connections in precast construction, particularly where precast piles or columns are used in conjunction with precast pier caps and/or pile caps. To accomplish this goal, a brief summary of connection types commonly encountered will be presented, and critical design considerations will be summarized. Finally, examples of connections which are currently in use will be highlighted, and typical design problems presented and solved.

There is increased use of precast pile caps and bent caps in construction, and a wealth of details available for use. In many cases the economy of precast construction is evident, as in marine work, where concrete delivery is often difficult or impractical, and in fast-track construction, where Maintenance of Traffic or other schedule considerations warrant an approach which minimizes on-site construction. These considerations alone justify development of a generalized design approach for these types of connections.

However, it has also been the experience of the author that in many cases details are proposed without sufficient consideration of the forces in play at pile to cap connections – whether precast construction is the preferred choice or not. On one hand a key example is the use of tension connectors where moment transfer is necessary (unconservative design), and on the other hand where fixed connections are proposed to develop the full strength of the pile or cap despite a rigorous structural analysis which indicates the connection will see only a fraction of the flexural, axial and shear capacity of the pile and/or cap in use (conservative design).

Design Considerations

The design process must consider a number of factors, and a few are considered by the author to be of almost universal utility. The factors considered most important will be summarized below - though some may seem to be a digression from the subject at hand. However, it is ultimately necessary to accurately assess the forces which act at the pile to cap interface, or which must at least be considered for structural stability. Once given the forces acting at the pile to cap connection the design process can be simplified to the following steps: 1) Design for the forces at the interface of the pile and cap, and 2) Determination that the reinforcement provided at the pile to cap interface is adequately developed above and below the interface.

Soil-Structure Interaction Basics

One of the first considerations which must be made in the design of a foundation regards pile behavior (which is where I digress from pile to cap design): Is the pile adequately embedded to develop fixity in the soil, and is the embedment in the soil sufficient for axial capacity? These questions must be answered in order to properly establish the forces acting on the pile – at the head of the pile and below in the soil mass.

There are three terms which are especially useful with respect to pile embedment: The Depth to Fixity, the Minimum Tip Elevation, and the Anticipated Tip Elevation. The Depth to Fixity is the depth below grade at which the pile moment is greatest. It is useful in analysis to establish the Depth to Fixity, since it can be used in structural analysis as the depth below grade at which the pile can be fixed so as to produce the approximate forces in the substructure. The Minimum Tip Elevation is the depth below grade to which a pile must be driven (or otherwise installed) so that it is fixed in the soil mass. The Minimum Tip Elevation will always be greater than the elevation established for the Depth to Fixity, since it is necessary to firmly anchor the pile in the soil below the Depth to Fixity. Finally the Anticipated Tip Elevation is the depth at which axial capacity is obtained based on established design loads. The Anticipated Tip Elevation is necessary to prevent foundation settlement and thus redistribution of load. These definitions are illustrated in Figure 1. These terms are important, because without proper embedment, the actions in the pile (and in particular at the pile to cap interface) cannot be established accurately. Without proper embedment, i.e., pile fixity in the soil mass, the moment at the pile to cap interface may be as much as twice the value calculated for a pile firmly embedded and fixed in the soil mass. The Depth to Fixity is often assumed to be on the order of 5 to 10 feet below grade for precast piles typically in use (12 to 30 inch square precast prestressed piles): It is deeper for large stiff piles and for soft weak soils, and shorter for flexible piles and stiff soils. It has a direct correlation to the 'Characteristic Length' of the pile. Programs like FB-Pier, which run P-Y analysis of soil-structure interaction, provide excellent means for direct calculation of the Depth to Fixity and Minimum Tip Elevation.

Pile to Cap Connection Basics

Pile to cap connections may be pinned or fixed. A fixed connection is capable of developing the maximum anticipated forces at the pile to cap interface, not necessarily the development of the full moment capacity of the pile or cap - which is often too conservative an approach. A pinned connection is free to rotate at the pile to cap interface, or hinged so that the connection capacity is less than that required of a fixed connection, but without creating a structural instability. In general a pinned connection develops a degree of fixity, and reinforcement in a pinned connection may yield, effectively creating a plastic hinge.

A fixed connection is generally called upon to carry both tension and flexure. And, in general, fixity requires a great deal more reinforcement than design of a pinned connection sized for the same tension force. Even a relatively small moment will increase the required tensile steel. Review of the free body diagrams in Figure 2 shows the difference schematically, and Figure 3 provide the interaction diagram representative of the strength envelope for the pile shown schematically in Figure 2. The results of an analysis of pure tension indicates the phi factored tensile capacity of the connection is approximately 64 kips, while an analysis of moment at zero axial load indicates a phi factored flexural capacity of 36 kip-ft. Inspection of the interaction diagram for the 1 inch dia. threadbar shows that between these two points it is possible to estimate any combination of moment and tension using linear interpolation between the two extremes. The graphic output shown in Figure 3 was obtained from the program FB-Pier, which in addition to running a P-Y analysis of the soil-structure interaction can provide an ultimate strength check of the connection reinforcement. I include this example because it has been my experience that moments used to size pile reinforcement often occur at the head of a pile (at the pile to cap interface), but are sometimes assumed to occur in the soil mass below. It

is necessary to have a firm handle on both the magnitude and location of the loads which can occur in the pile.

The design considerations summarized above are important, but yield little information useful in the design of a connection. The remainder of this section addresses criteria directly applicable to design of pile to cap connections.

Fixity through Pile Embedment

The Florida Department of Transportation recognizes a rule of thumb useful for design of pile to cap connections. It is applicable to typical precast piles, i.e., those in the range of 12 to 30 inches square. Experiment and experience have shown that piles of this size are fixed in a cap if embedded a minimum of four (4) feet. And by inference, without further calculation, it is acceptable to take 25% of the pile flexural capacity for a pile embedded one (1) foot into a cap, or 50% of its flexural capacity for a pile embedded two (2) feet into the pile cap. This rule has been used successfully for many years. And though somewhat awkward when using precast components, it can be useful for precast pile to cap connections. One example of such use is for the Sanibel Island Causeway, where rather than redesigning the pile to pile cap connection details in the contract plans, precast pile caps were designed with four (4) foot recesses and placed directly over precast 30" square piles. The connection was finished by placement of a fluid grout in the recess between the piles and cap. This connection detail is shown schematically in Figure 4. It is interesting to compare the development length of strand in the pile to the four (4) foot rule. According to Equation 5.11.4.2-1 of the AASHTO LRFD Bridge Specifications the development length for a 24 to 36 inch prestressed pile using ½ inch lo-lax strand will be in the range of 5 to 6 feet, considerably longer than the 4 foot pile embedment necessary for complete fixity. It is likely however, that embedment of a pile in a cap provides confining pressures which reduce the required development length of strand.

Design for Fixity without Pile Embedment

Four foot embedment of piles in pier or pile caps is not always practical, simply because most caps will be less than four feet deep, and because pile embedment interferes with placement of rebar in a cap or pier, necessitating the rebar be placed between piles. Strut and tie analysis of deep beams and footings indicates the preferred placement of tensile steel in the bottom mat is directly over piles, in line with the path of tension ties (principle tensile stresses) and intersecting compression struts (principle compressive stresses). Consequently, it is generally preferable to anchor a pile to a cap using a steel cage which crosses the pile to cap interface. An analysis is generally needed to confirm that the strength of the rebar cage at the pile to cap interface is sufficient to handle the anticipated design loads. Alternately, as in the case of design emulation (where a precast system is substituted for C.I.P. system), an analysis is conducted to confirm that the strength of the precast connection is of equal strength to that of the original C.I.P. design. The former is done by comparing factored loads to the ultimate strength interaction diagram for the section at the pile to cap interface, and the latter by comparing the strength interaction diagram of the alternate precast connection to that of the C.I.P. connection at the critical section(s). There are a number of programs available for this purpose. FB-Pier has a subroutine for calculating the ultimate strength interaction diagram at any section in the pile or column, and the Florida DOT Web Site has a free MathCAD template capable of generating

biaxial interaction diagrams for both reinforced and prestressed sections. Hand calculations for interaction diagrams are possible, but tedious. Simply put, the strength of the connection is obtained by generation of the ultimate strength interaction diagram at the critical section.

A second set of analyses is needed to confirm that the pile to cap reinforcement is adequately embedded and developed above and below the critical section at the pile to cap interface. In reality development of the pile to cap reinforcement below the critical section often relies on a different set of parameters than development of the connection above the critical section, thus two analyses are often required. There are various mechanisms used to develop the connection above and below the pile to cap interface, and it is likely that more than one mechanism will have to be employed and checked to verify adequate development. Mechanisms for development of pile to cap connections are discussed below.

Pile to cap connections may consist of rebar cages, Dywidag or Williams high strength threadbar – either post-tensioned or unstressed, flanged pipe sections, strand anchorages, grout sleeve splices, or any of a number of other proprietary systems. We will confine this paper to the first three types; rebar cages, high strength threadbar, and flanged pipe sections. There are multiple reasons to group these connection types together; chief among these is that the connection details used with these systems don't require use of templates or require precise alignment of connected elements – whereas grout sleeve splices and other proprietary connections often do. Just as important, these types of connections have proven to be very economical. Finally, variations of these connection types have found use not only in several major bridge projects recently completed, but also in small residential and commercial projects, indicating the suitability of their use for a wide range of structure types.

What are the mechanisms employed for development and anchorage of these types of connections? Development through bond of rebar to concrete and grouts is one mechanism, development through bond of strand to concrete is another type of mechanism, and mechanical anchorage using nuts and washers or combinations of nuts and bearing plates to form truncated shear cone failure surfaces is another. And by design, these mechanisms often act in combination with one another. All of these mechanisms are covered by Code; both ACI 318 and the AASHTO LRFD Bridge Design Specifications provide equations for strand and rebar development. Appendix D of ACI 318 provides a fairly comprehensive discussion of truncated shear cones, as does the PCI Design Handbook. A comprehensive design approach can be obtained by careful application of the various Code equations and design criteria. A quick look at one of the simplest types of connections available will serve to illustrate the utility of a design approach which can be applied to connections of virtually any size or capacity.

A Simple Pile to Cap Connection Detail

Refer to Figure 5, which shows a 14 inch square precast pile with a single high-strength threaded rod embedded in the center of the pile. The pile is connected to a precast cap, 2 feet square in section, which rests on the pile and incorporates an 8 inch circular void centered directly over the threadbar. Once the cap is set the void is filled with structural grout. This connection provides ample tolerance for misdriven piles, given a fairly standard driving tolerance of +/- 3 inches. It provides an approximate phi factored moment capacity of 35 kip-ft based on the pile interaction

diagram for a single 1" high-strength threadbar centered in a 14" square pile (assuming a limiting stress in the bar of $.6 \times F_{pu}$ or 90 ksi), or phi factored tensile capacity of approximately 64 kips .

Development is obtained in the cap above the pile to cap interface via a truncated cone failure surface under the nut fully engaged on the threadbar. Refer to Equation 6.5.3 in the PCI Design Handbook, 4th Edition, for the strength available based on the truncated shear cone failure surface. This analysis is straight forward, and does not attempt to take into account any reinforcement in the cap.

Development of the threadbar in the pile is more complicated: The bar is grouted into a corrugated metal pipe sleeve, and the pipe sleeve is, in turn, bonded to the pile concrete. (The use of a corrugated metal pipe sleeve allows the pile to be driven without interference from the threadbar extension). The depth of the pipe sleeve must satisfy two conditions: First, it must be long enough to ensure that the bond between threadbar and grout in the corrugated pipe is adequate to resist the bar tension, and also long enough to ensure that the bond between the corrugated pipe and concrete is adequate to resist the same tension; Second, it must be long enough to ensure that the pile strand is adequately developed a sufficient distance above the base of the threadbar and pipe sleeve to effectively create a non-contact splice between the pile to cap connection and the pile strand.

Again, refer to Figure 5. As stated above, Equation 6.5.3 in the PCI Design Handbook, 4th Edition, provides an estimate of the load associated with a truncated shear cone failure at the base of a headed stud (In this case a threadbar with nut engaged at the base of the threadbar). See Figure 6.5.6 Case 1 in the PCI Handbook. For a 14 inch square pile using 6 ksi concrete (ignoring the strand in the pile) the load which will cause the pile to crack along a truncated shear cone failure surface is between 41 and 48 kips (41 kips includes a phi factor of .85 – 48 kips does not). A clear mechanism for development of the truncated shear cone is a nut engaged at the base of the threadbar. Where the threadbar is encased by a corrugated pipe, the mechanism is less evident. However, by virtue of bond stress between the corrugated pipe (or threadbar) and concrete, the corrugated pipe (or threadbar) would have to extend approximately 18" below the top of an 'equivalent' nut in order to generate the same failure stress in the concrete. (Bond strength between bars of virtually any diameter and concrete of strength f'_c [psi] is estimated to be $.035 \times \sqrt{f'_c}$ in kips per inch. ¹) The strand above the failure surface must be developed sufficiently to resist the available moment capacity of the pile to cap connection (or threadbar tension). For the 14" pile with a single 1" diameter high strength threadbar centered in the pile, the available moment capacity is approximately 35 kip-ft, as stated above. For the 14" square pile with (8) ½" lo-lax strand the cracking moment at the transfer length is 62.5 kip-ft, so the threadbar embedment of 48 inches is seen to be a conservative value. It is greater than the 30 inch transfer length x 35 kip-ft/62.5 kip-ft + the 18 inch bond length below lowest possible failure surface + the 4 inch vertical dimension from the base of the assumed failure surface to the location where the assumed failure surface cuts the strand. A similar set of calculations for strength and development is applicable to much more robust pile to cap connections, and is equally applicable to connections using rebar, threadbar, or flanged pipe sleeves. In the worked example at the end of this paper, calculations are produced using a MathCAD template for a pile to cap connection between a 54 inch cylinder pile and precast bent cap.

Typical Connection Details

To provide details of the types of connections being discussed in this paper the following figures are provided.

Figure 6 illustrates several robust connection details, ranging from flanged pipe segments, to rebar or threadbar cages dropped into pile voids, to cages precast into cap beams. In this figure a cylinder pile is shown, but pile voids are often cast into 24, 30 and 36 inch square piles as well. A void of 18" is commonly used in 30 inch square piles, whereas the 36 inch square pile is typically cast with a 22 ½ inch void.

Figure 7 shows connection details similar to the one discussed above and shown in Figure 5. This connection employs corrugated metal pipe embeds and either Grade 60 rebar, or threadbar with a yield ranging from 75 to 120 ksi. These connections are very commonly used, though more often for tension connections in pinned piles, than for moment connections in fixed piles. They are (arguably) better suited to smaller precast piles - 12 inches to 18 square inches - than to larger diameter piles which can be fabricated with large voids suitable for use with rebar or threadbar cages. Though Figure 7 shows a connection employing a single high-strength bar, use of multiple bars in square or round bolt patterns can be used to provide higher tensile and flexural capacities. The primary caveat is that voids provided in the pier caps must be larger than 8 inches in diameter to fit over the bar group.

The State of Texas has recently adopted a precast cap to column detail similar to the concept shown in Figure 7. It employs corrugated metal pipe embeds in the precast cap, and is spliced to rebar extending out of the top of C.I.P. columns. Though similar in appearance to the detail shown in Figure 7, these standard details are not appropriate for pile to cap connections. They require very tight tolerances between the pier cap and columns. These tolerances are almost impossible to obtain when dealing with driven piles.

It is noteworthy that use of high-strength threadbar, as opposed to standard Grade 60 rebar, allows cage reinforcement to accommodate very large moments. Threadbar yield strengths typically range from 75 ksi to 120 ksi. (A-722 steel has an ultimate tensile capacity of 150 ksi and an approximate yield of 120 ksi). Though the ACI Code Commentary in Appendix D of ACI 318 stipulates yield strengths in excess of 100 ksi may not be suitable for use with current development length equations, mechanical anchorage appears to circumvent this restriction, since bond is not the mechanism for development and anchorage.

Example Projects

Examples of recently completed precast projects using construction details discussed in this paper are presented in this section. Projects include the Sanibel Island Causeway, St. George Island Bridge, Escambia Bay and Twin Spans Bridges, as well as two smaller projects, one a residential dock and the other a municipal pier. These projects highlight the utility and versatility of the pile to cap connections discussed.

The Sanibel Island Bridge Replacement Project was mentioned earlier in conjunction with fixity obtained via pile embedment. Photos of the fabrication and erection of the cap are shown in Figure 8. Voids (4 foot recesses) were precast into the pile cap, which was cast on a barge due to limited site access. The cap was later set on a group of 24 inch piles, and then grouted in place.

The St. George Island Design-Build Project employed a number of innovative details. The pile caps, though not precast, were cast in precast tubs, and fixed to cylinder piles using a connection similar to that shown in Figure 6. See Figures 9 & 10. For low level piers precast caps were set on cylinder piles, then flanged pipe sections were dropped through the cap voids and into the pile voids. The connection was not grouted until girders had been set, allowing very rapid erection of the low level structure. This process is shown schematically in Figure 11.

Both the Escambia Bay I-10 Bridge (FDOT District 3) over Escambia Bay east of Pensacola, and the Twin Spans I-10 Bridge (LADOT) over Lake Pontchartrain outside of New Orleans utilized precast caps and 36 inch square voided piles. They employ very similar pile to cap connection details, as shown in Figures 12 through 14. Rebar cages were dropped into the pile void and grouted in place. Caps were then set over the piles, with the rebar cages extending up into the pier cap void. The connection was completed by making a secondary pour in the pile cap.

Finally, Figures 15 and 16 illustrate use of the detail shown in Figures 5 and 7 for a private dock and municipal pier. These projects illustrate the use and economy of a single threadbar connector to develop moment fixity between small precast piles and caps.

Design of a Fixed Connection between a 54 Inch Cylinder Pile and Bent Cap

A set of design calculations for a precast pile to cap connection is presented below. These details were recently proposed for a major bridge project as a Value Engineering alternate. The proposed structure is being built in a marine environment, over shallow bay waters. This set of calculations is intended to provide an example of design emulation, where, rather than deriving loads from scratch, a precast cap and pile to cap connection are sized to be of equal strength to the C.I.P. cap and connection shown and detailed in the Contract Documents.

The C.I.P. pier cap measures 62 feet long by 48 inches deep and 60 inches wide. It is fixed to 54 inch cylinder piles using (16) #11 bars in a 35.875 inch diameter bolt circle. See Figure 18. Rather than derive design loads from scratch, a precast cap and pile to cap connection were proposed which would emulate the original design – providing equal or greater strength to the existing design while preserving the proposed dimensions of the structure.

The primary difference between the precast and C.I.P. option is the incorporation of a void through the precast cap over each pile to accommodate a threadbar cage dropped through the cap and into the piles. To emulate the existing design, the precast cap was designed with the same gross cross section, 48 inches deep by 60 inches wide. An hour glass void, similar to that used on the Escambia Bay Bridge Replacement was sized, having a minimum diameter of 24 inches and maximum diameter of 26 inches. The void forces relocation of the steel in the cross section.

The alternate precast section is shown in Figure 19. Calculations show it has strength equal to that of the existing design.

High strength threadbar was sized for the pile to cap connection. It could not be identical to the C.I.P. design since the bolt circle had to fit within the precast void, while providing clearances for concrete consolidation. A bolt circle 17.5 inches in diameter was chosen, using (11) 1 ¾ inch diameter Grade 75 threadbar. Comparison of the interaction diagrams confirms it's strength to be on par with that of the existing connection.

The following calculations recap the design comparisons summarized above, and include calculations for the development of the connection above and below the pile to cap interface.

Design Emulation
 Precast Alternate to a C.I.P. Pier Cap

Cap Design

Approximate Flexural & Shear Capacity of the C.I.P. Pier Cap

$$h_{\text{cap}} := 48 \quad b_{\text{cap}} := 60 \quad A_{s1} := 7.156 \quad A_{s2} := 2.31$$

$$y_{s1} := 43.6875 \quad y_{s2} := 37.125$$

$$f_c := 5 \quad \text{ksi} \quad \text{C.I.P. Concrete Strength}$$

$$f_y := 60 \quad \text{ksi} \quad \text{Rebar Yield Strength}$$

$$\phi := .9$$

$$d := \frac{A_{s1} \cdot y_{s1} + A_{s2} \cdot y_{s2}}{A_{s1} + A_{s2}} \quad a := \frac{(A_{s1} + A_{s2}) \cdot f_y}{.85 \cdot f_c \cdot b_{\text{cap}}}$$

$$M_d := \phi \cdot (A_{s1} + A_{s2}) \cdot f_y \cdot \left(d - \frac{a}{2} \right) = 2.616 \times 10^4 \quad \text{kip - inches}$$

$$V_c := \frac{2 \cdot \sqrt{f_c} \cdot 1000 \cdot b_{\text{cap}} \cdot d}{1000} \quad \phi \cdot V_c = 330.938 \quad \text{kips}$$

$$A_v := 2.44 \quad s := 12$$

$$V_s := \frac{A_v \cdot d \cdot f_y}{s} \quad \phi \cdot V_s = 171.606 \quad \text{kips}$$

$$\phi \cdot (V_c + V_s) = 502.544 \quad \text{kips}$$

Approximate Flexural and Shear Capacity of the Precast Alternate

$$f_{\text{calt}} := 6 \quad \text{ksi}$$

$$A_{s1\text{alt}} := 4.156 \quad A_{s2\text{alt}} := 4.156$$

$$y_{1\text{alt}} := 42.875 \quad y_{2\text{alt}} := 36.875$$

$$d_{\text{alt}} := \frac{A_{s1\text{alt}} \cdot y_{1\text{alt}} + A_{s2\text{alt}} \cdot y_{2\text{alt}}}{A_{s1\text{alt}} + A_{s2\text{alt}}} \quad a_{\text{alt}} := \frac{(A_{s1\text{alt}} + A_{s2\text{alt}}) \cdot f_y}{.85 \cdot f_{\text{calt}} \cdot b_{\text{cap}}}$$

$$M_{dalt} := (A_{s1} + A_{s2}) \cdot f_y \cdot \left(d_{alt} - \frac{s_{alt}}{2} \right) = 2.676 \times 10^4 \quad \text{kip - inches}$$

$$V_{calt} := \frac{2 \cdot \sqrt{f_{calt}} \cdot 1000 \cdot b_{cap} \cdot d_{alt}}{1000} = 370.645 \quad \text{kips}$$

$$A_{valt} := 4.31 \quad s_{alt} := 13.5$$

$$V_{salt} := \frac{A_{valt} \cdot d_{alt} \cdot f_y}{s_{alt}} = 219.756 \quad \text{kips}$$

$$\phi \cdot (V_{calt} + V_{salt}) = 531.36 \quad \text{kips}$$

The Precast Alternate utilizes 6 ksi concrete and (8) #11 T & B reinforcement. Shear reinforcement is provided by (4) #5 bars spaced 15 inches on center.

By Comparison the C.I.P. Cap utilizes 5 ksi concrete and (7) #11 T & B, but has (2) additional longitudinal #5 bars in the facia reinforcement. Shear reinforcement is provided by (2) #5 bars spaced 12 inches on center.

Pile to Cap Connection

Summary of Interaction Diagram Ultimate Flexural Strength, for comparison of the Pile to Cap Connection Details.

Common to both alternates is the 54 inch Diameter Cylinder Pile. The cylinder pile uses (24) tendons, each tendon comprised of (2) 1/2 inch strand. The moment capacity of the cylinder pile is needed to establish the development length of the pile to cap connection below the pile to cap interface.

The pile to cap connection in the Contract Documents utilizes (16) #11 bars in a bolt circle 35.875 inches in diameter. This provides the target flexural capacity of precast alternate design.

The alternate pile to cap connection utilizes (11) 1 3/4 inch diameter Grade 75 threadbar, in order to provide the same or greater capacity as the original design.

FDOT Biaxial Interaction Diagram Program Output:

$$\phi M_{n54} := 3.6 \cdot 10^4 \quad \text{kip - inches} \quad @ P_n := 250 \quad \text{kips} \quad \phi_{54} := 1.0$$

$$f_{c54} := 7 \quad \text{ksi} \quad f_{pu} := 270 \quad \text{ksi}$$

$$\phi M_{nCIP} := 2.6 \cdot 10^4 \quad \text{kip - inches} \quad @ P_{\text{allow}} := 250 \quad \text{kips} \quad \phi_{CIP} := 0.9$$

$$f_{cCIP} := 5 \quad \text{ksi} \quad f_{\text{req}} := 60 \quad \text{ksi}$$

$$\phi M_{nPrecast} := 3.45 \cdot 10^4 \quad \text{kip - inches} \quad @ P_{\text{allow}} := 250 \quad \text{kips} \quad \phi_{Precast} := 0.9$$

$$f_{cPrecast} := 6 \quad \text{ksi} \quad f_{\text{req}} := 75 \quad \text{ksi}$$

It can be seen that the precast pile to cap connection provides strength greater than or equal to that of the CIP design which is being emulated. These three values are useful in determining development length requirements above and below the pile to cap interface.

Above the pile to cap interface the connection strength will be developed by the formation of a truncated shear cone failure surface. Reference the PCI Publication 'Design & Typical Details of Connections for Precast and Prestressed Concrete: Figure 4.11.4 Case 3'. See calculation below:

The threadbar in the precast design need only develop a tensile stress of 60 ksi to provide moment capacity equal to that of the CIP connection detail. This can be verified by running the interaction diagram for the threadbar connection at the reduced stress of 60 ksi (as opposed to the yield stress of 75 ksi).

To provide anchorage above the pile to cap interface, it is sufficient to show that the truncated shear cone failure surface will develop the entire tensile force in the threadbar at the required stress of 60 ksi.

$$\phi_{\text{tensile}} := 0.9 \quad f_{ps} := 60 \quad \text{ksi} \quad A_s := 11.2405$$

$$\phi_{\text{tensile}} \cdot f_{ps} \cdot A_s = 1.429 \times 10^3 \quad \text{kips} \quad \text{Required Tensile Force to be Developed by Truncated Shear Cone Failure Surface.}$$

$$\phi_{\text{cone}} := 0.85 \quad f_{\text{req}} := 6 \quad \text{ksi}$$

$$d_{\text{eff}} := 12.375 \quad l_{\text{embed}} := 42 \quad \text{inches}$$

$$P_c := \frac{4 \cdot \sqrt{f_c} \cdot 1000 \cdot b_{\text{cap}} \cdot (d_{\text{eff}} + 2 \cdot l_{\text{embed}})}{1000} = 1.792 \times 10^3 \quad \phi_{\text{cone}} \cdot P_c = 1.523 \times 10^3 \quad \text{kips}$$

It is apparent that the tensile strength of the truncated shear cone failure surface exceeds the tensile force which will be developed by the pile to cap connection at the maximum anticipated tensile stress in the threadbar.

Below the pile to cap interface, the threadbar must effectively lap with the strand in the cylinder pile. The required depth of embedment of the threadbar in the pile can be estimated using the same approach taken to determine the embedment of the single threadbar in a 14 inch pile. Refer again to Figure 7. A truncated shear cone failure surface is assumed to form some distance below the pile to cap interface, consistent with development of tensile stress in the pile sufficient to crack the pile in the absence of strand. The failure surface is precipitated by mechanical anchorage of the threadbar. The strand must be anchored above this failure surface. Were the pile to develop it's full moment capacity the strand would have to extend the full development length of the strand at nominal flexural capacity. As the moment in the connection is less than the nominal flexural capacity, the stress developed in the strand is a fraction of the nominal stress at ultimate strength, and the developement length required can be reduced proportionately.

$K := 1$	$f_{psnominal} := 265$	ksi	Estimated Stress in the Strand at Nominal Flexural Capacity
$f_{pe} := 170$	ksi	Calculated Effective Stress after Losses	
$d_b := 0.5$	inches		

$$l_d := K \cdot \left(f_{psnominal} - \frac{2}{3} \cdot f_{pe} \right) \cdot d_b = 75.833 \quad \text{inches}$$

$$\phi M_{nCIP} = 2.6 \times 10^4$$

$$\phi M_{n54} = 3.6 \times 10^4$$

$$\frac{\phi M_{nCIP}}{\phi M_{n54}} = 0.722$$

It is assumed that the stress in the pile at the capacity of the CIP pile to cap connection is the nominal stress at ultimate strength times the ratio of the two moments shown

$$f_{ps} := f_{psnominal} \cdot \frac{\phi M_{nCIP}}{\phi M_{n54}} = 191.389 \quad \text{ksi}$$

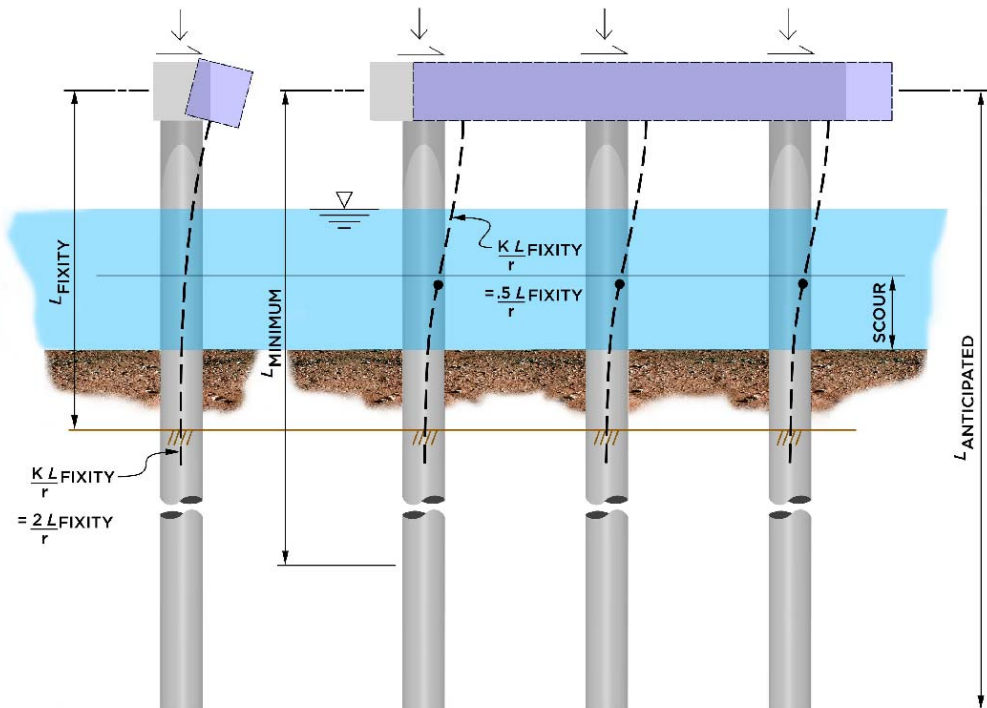
$$l_{transfer} := 30 \quad \text{inches}$$

$$l_{adjusted} := l_{transfer} + \frac{f_{ps} - f_{pe}}{f_{psnominal} - f_{pe}} \cdot (l_d - l_{transfer}) = 40.319 \quad \text{inches}$$

Refer to Figure 20. The minimum embedment of the threadbar cage in the pile is seen to be approximately 58.82 inches. In no case would the required embedment be greater than 96 inches based on the full development length of 1/2 inch strand.

REFERENCES

1. 'Design of Concrete Structures' Winter and Nilson, 9th Edition
2. AASHTO LRFD Bridge Design Specifications, 5th Edition
3. ACI 318-05 'Building Code Requirements for Structural Concrete'
4. PCI Design Handbook, 4th Edition



- LENGTH TO FIXITY: L_{FIXITY} = THE LENGTH OF THE PILE/COLUMN FROM THE CAP TO THE POINT OF MAXIMUM MOMENT IN THE SOIL MASS.
- MINIMUM TIP ELEVATION: $L_{MINIMUM}$ = THE MINIMUM LENGTH OF THE PILE TO ENSURE SUFFICIENT EMBEDMENT FOR LATERAL STABILITY.
- ANTICIPATED TIP: $L_{ANTICIPATED}$ = THE LENGTH OF THE PILE ESTIMATED TO PROVIDE THE REQUIRED AXIAL CAPACITY.

Figure 1: Definition of Terms Descriptive of Soil-Structure Interaction Useful in the Analysis and Design of Pile Supported Structures. The Depth to Fixity is Always Less than The Minimum Tip Elevation, and the Anticipated Tip Elevation Must be Greater than or Equal to the Minimum Tip Elevation.

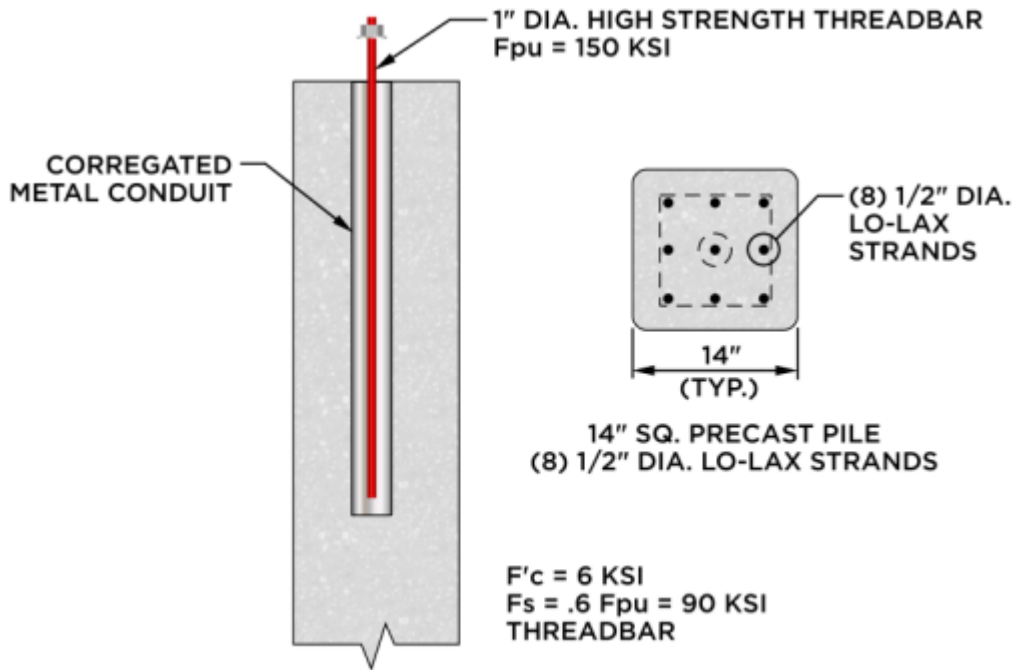
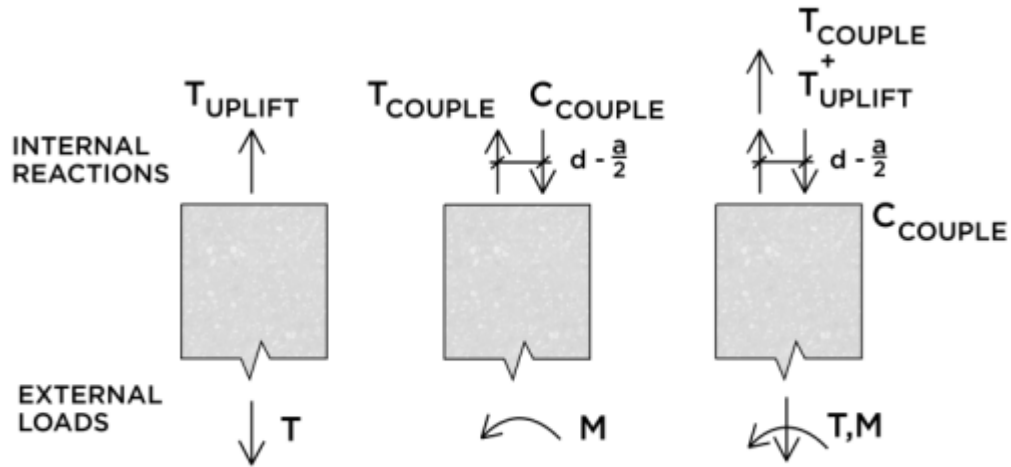


Figure 2: Comparison of a Tension Connection, Moment Connection, and Connection Combining Both Tension and Moment. See Interaction Diagram for the Above Referenced Pile in Figure 3.

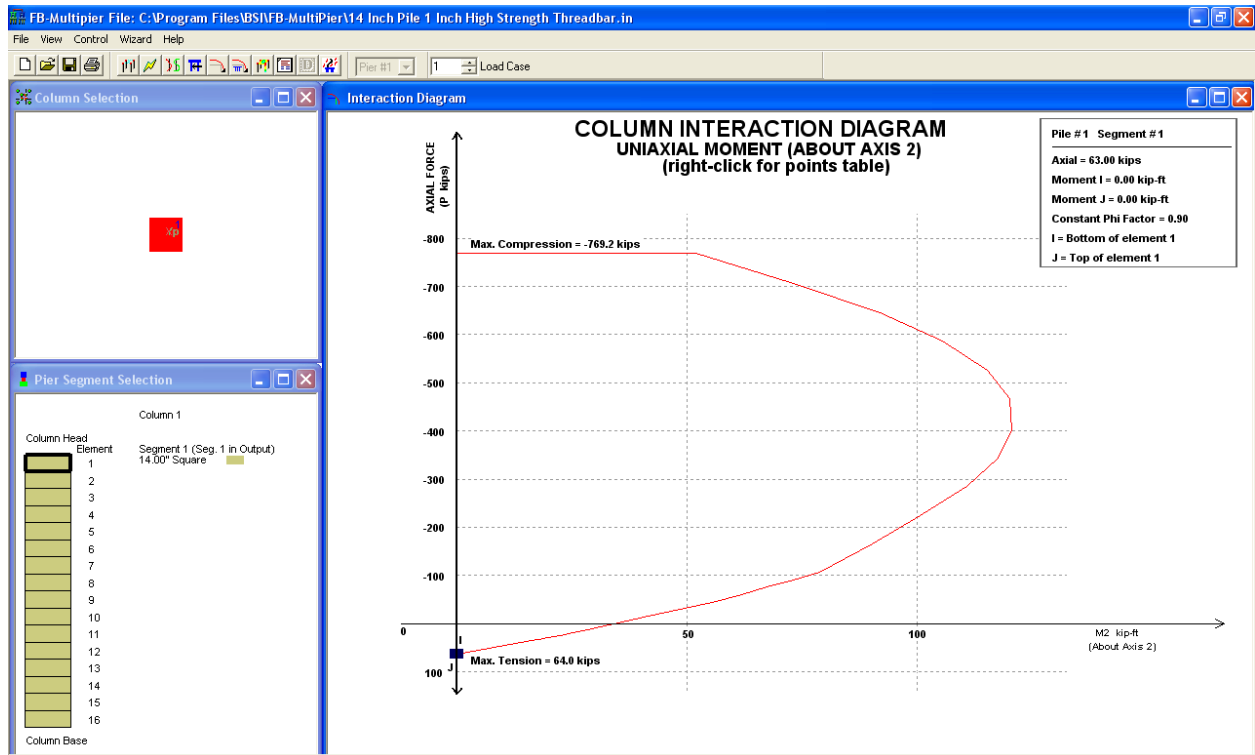


Figure 3: Uniaxial Interaction Diagram for the Pile Shown in Figure 2. A constant Phi Factor of 0.9 Applies to the Range Between Pure Tension (64 kips) and Pure Bending (35 kip-feet). For Any Combination of Moment and Tension Between these Limits Linear Interpolation is Appropriate.

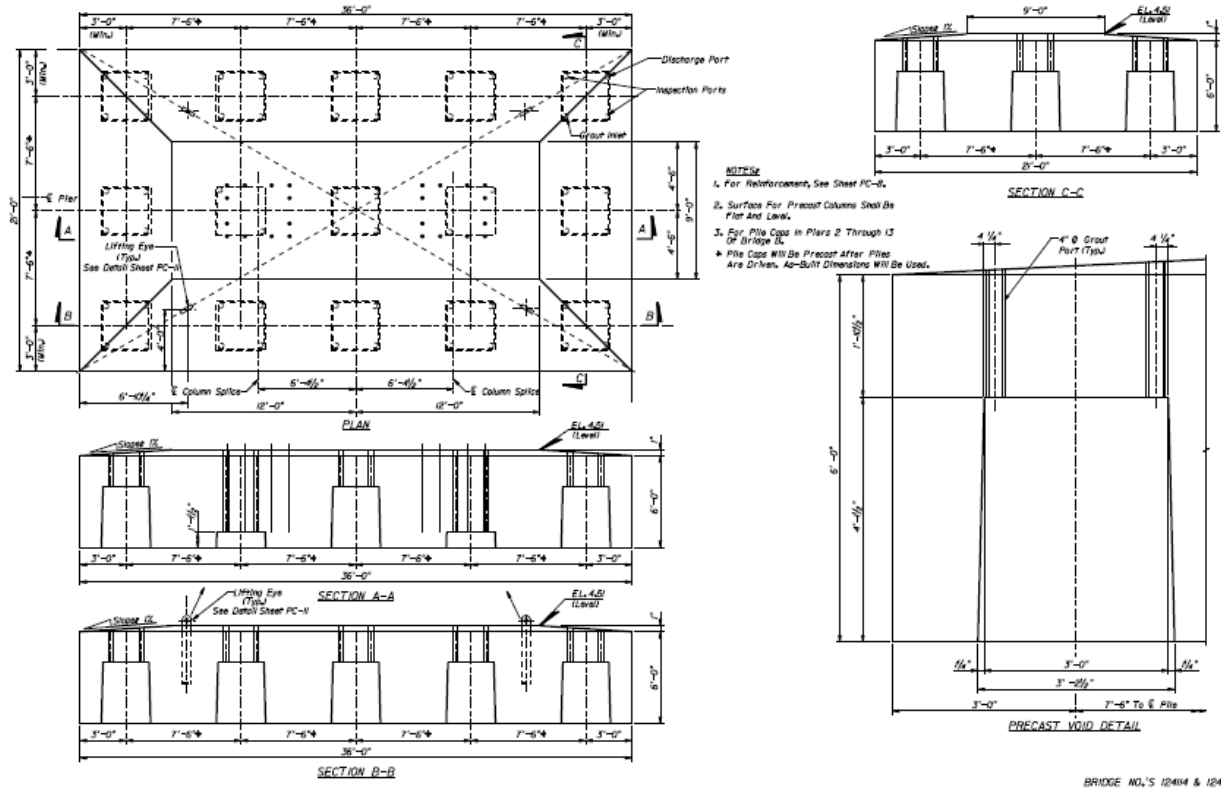


Figure 4: Details of the Sanibel Causeway Precast Pile Caps. The Precast Caps were Cast on Barges, Floated to their Final Location, then Lifted off the Barge and Set Over Driven Piles. Once in Place, Grout was Placed through Four (4) inch Diameter Grout Ports Fixing the Piles to the Caps.

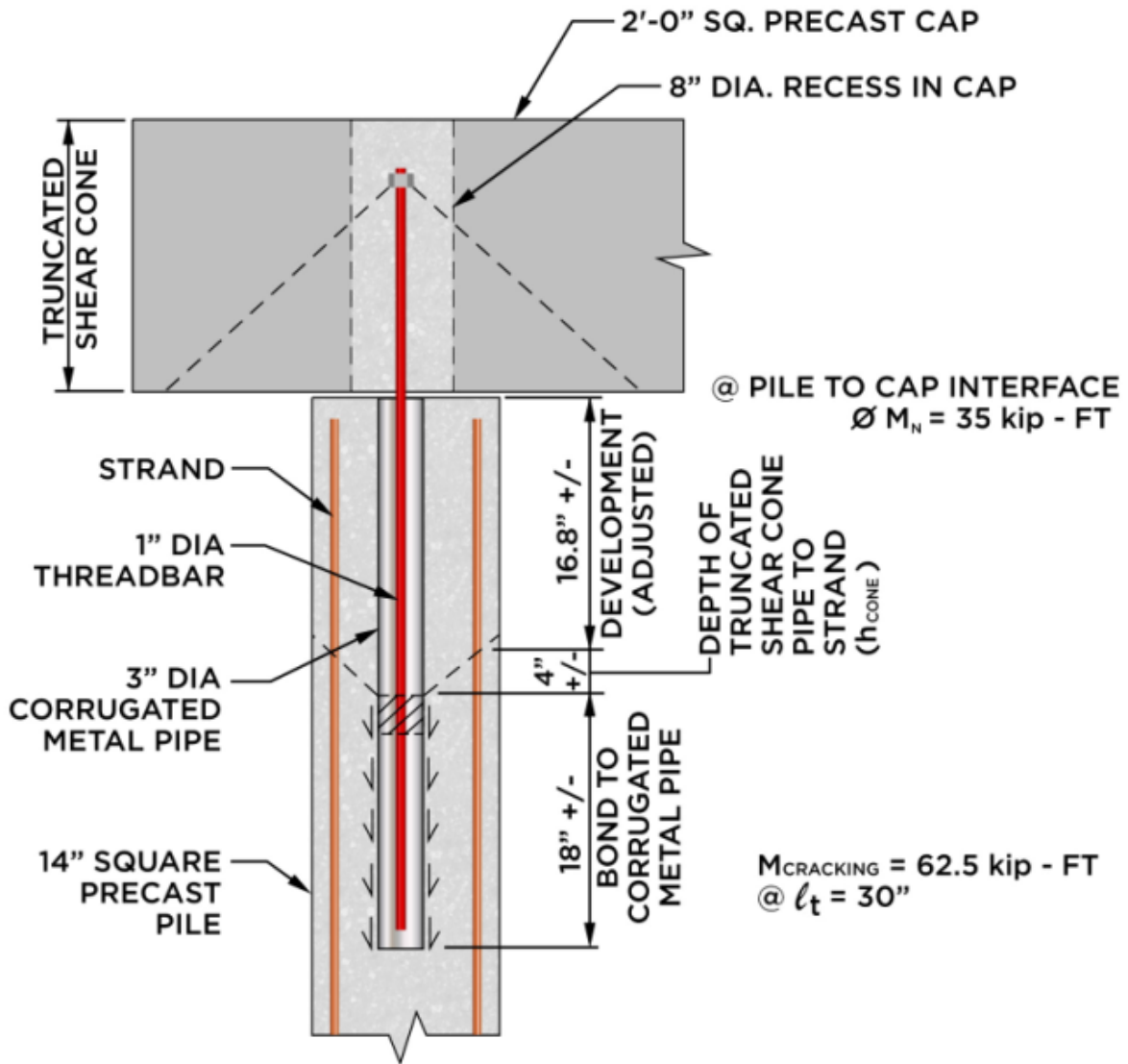


Figure 5: Schematic of a Pile to Cap Connection Illustrating the Assumed Development of the Threadbar Reinforcement in the Pile and Cap. In the Cap a Truncated Shear Cone Failure Surface is the Assumed Failure Mode. In the Pile the Connection Requires Development of the Strand in the Top of the Pile, and Threadbar within the Pile.

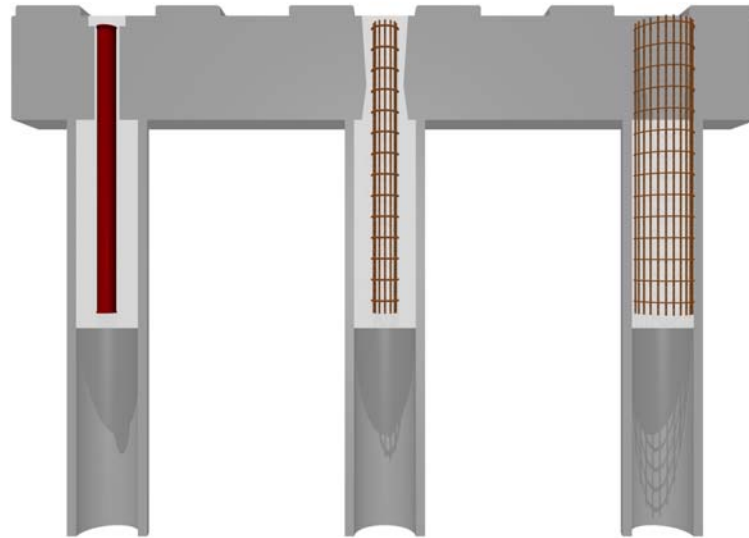


Figure 6: Three Different Types of Pile to Cap Connections are Shown: On the Left the Pile to Cap Connection Utilizes a Flanged Pipe Section Which is Dropped Through Voids Precast in the Pile and Cap. The Flanged Ends Provide Mechanical Anchorage. In the Middle, the Connection Utilizes a Rebar or Threadbar Cage. A Rebar Cage Generally Develops Development through Bond Stresses, While a Threadbar Cage Often Uses Mechanical Anchorage in the Form of Nuts and Washers, or Nuts and Bearing Plates. The Connection on the Right is Suitable for C.I.P. Caps or Caps formed within Precast Forms or Tubs, as Illustrated later for a Waterline Footing.

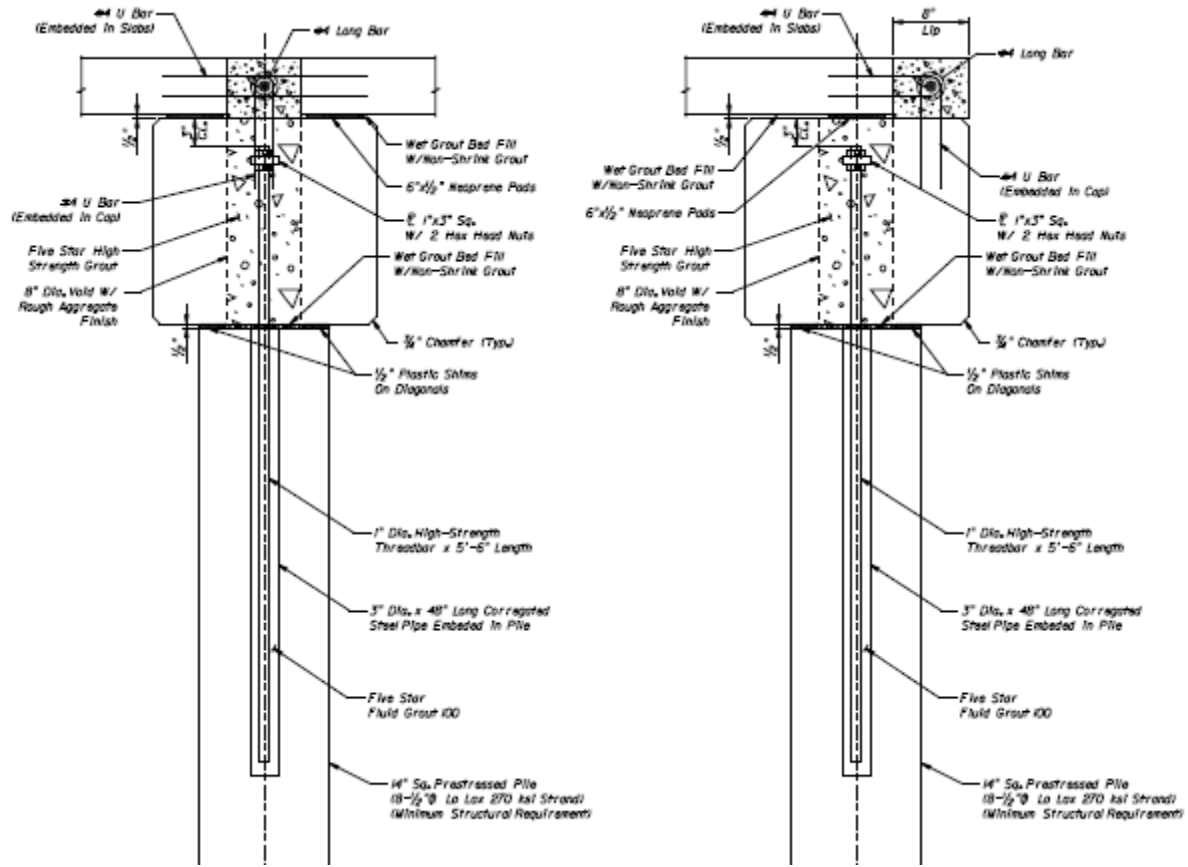


Figure 7: Examples of Threadbar Connections dropped into Corrugated Metal Conduit, then Grouted in Place. The Pile to Cap Connections Shown Above can be Modeled using the Approach illustrated in Figure 5.

Pile Caps are 36'-0" by 21'-0" by 6'-0"
& weigh in excess of 300 tons

Voids are formed for piles, which extend
four feet into the cap for fixity

Mass Concrete is monitored using Intellirock
temperature/maturity loggers, & insulated using
thermal blankets

Precast Caps are formed on Barges, lifted
from one barge to another for transport



Figure 8: The Sanibel Island Causeway in Lee County, Florida. Pile Caps were Precast on Barges, using Embeds to Form Four (4) foot Deep Voids in the Caps. Once the Cap had Cured, It was Transported by Barge to be Lifted and Set on a Group of 30 Inch Square Precast Piles. The Recess Between the Piles and Voids is Grouted Solid.

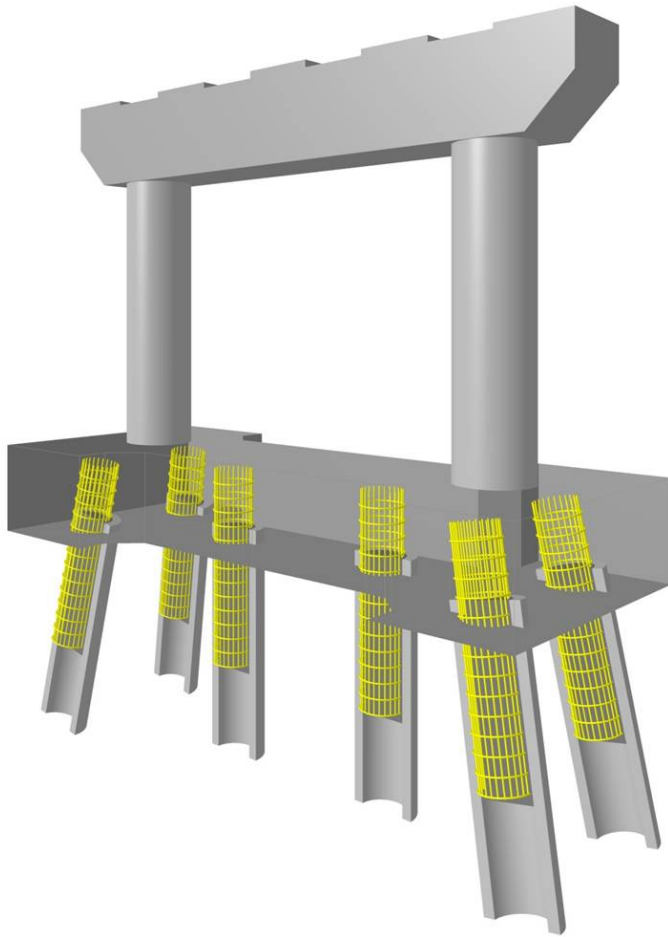


Figure 9: Pile to Cap Connection for a Waterline Footing. The Rebar Cage is dropped into the Pile Void and Grouted in Place. A Precast Form or Tub is then Suspended Off of the Piles and the Footing Cast in Place.

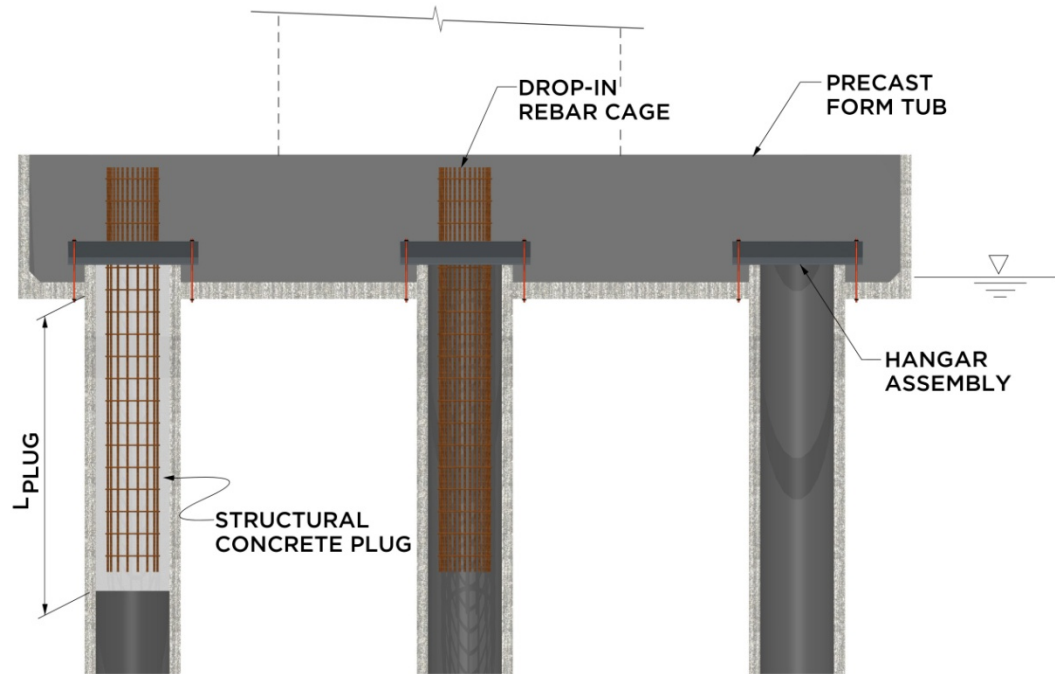


Figure 10: In this Figure the Precast Form 'Tub' is Shown and the Placement of the Rebar Cage and Hangers Suspending the Form in Place are Illustrated.

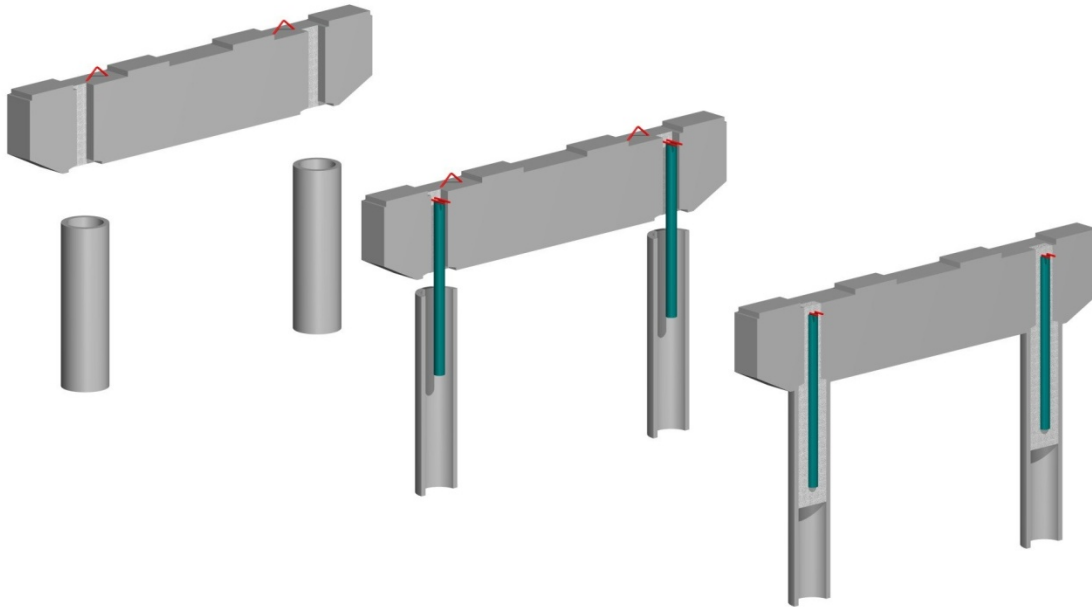


Figure 11: The low-level Piers of the St. George Island Bridge were constructed using Precast Caps and Cylinder Piles. The Precast Cap was set on the Cylinder Piles, after which flanged pipe sections were dropped through the Voids in the Cap and Pile. After Girders were set, the Void was filled with Structural Fill.



Figure 12: This Photo shows a Precast Cap Being Lifted off the Forms used to fashion the Voids for the Pile to Cap Reinforcement. The caps shown were used on the I-10 'Twin Spans' Bridge over Lake Pontchartrain.



Figure 13: Shown here is a Precast Cap being Lowered onto 36 Inch Square Precast Piles on the I-10 'Twin Spans' Project. Note the Spiral Wound Cage Reinforcement. AASHTO allows the Development Length of the Rebar to be Reduced by a Factor of 0.75 if a Spiral Cage is used (The Pitch must not exceed 4" and the Spiral Reinforcement must be at least 3/8 Inch in Diameter).

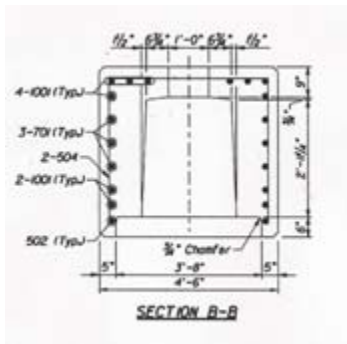
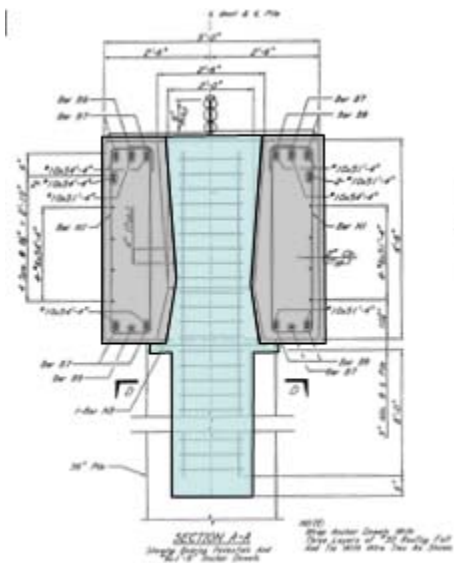


Figure 15: Two Different Pile Void Options are Shown: The Top Detail Utilizes an Hour Glass Void, and the Pile Sits under the Base of the Cap. This Detail was used on the Escambia Bay and Bay St. Louis Bridges. The Bottom Detail incorporates a Recess, so that the Pile fits into the Base of the Cap. It is the detail used on the 'Twin Spans' Bridge.



Figure 16: A Private Dock in Orange Beach, Alabama. It was constructed using the Pile to Cap Connection shown earlier in Figures 5 & 7. Total Precast Construction was used to complete this structure.



Figure 17: Shown here is the Garfield Ladner Memorial Pier in Waveland, Mississippi. The Pile to Cap Connection Detail is the same used on the Dock Shown in Figure 16. This Project employed precast panels which were ultimately Topped with 2 inches of Concrete.

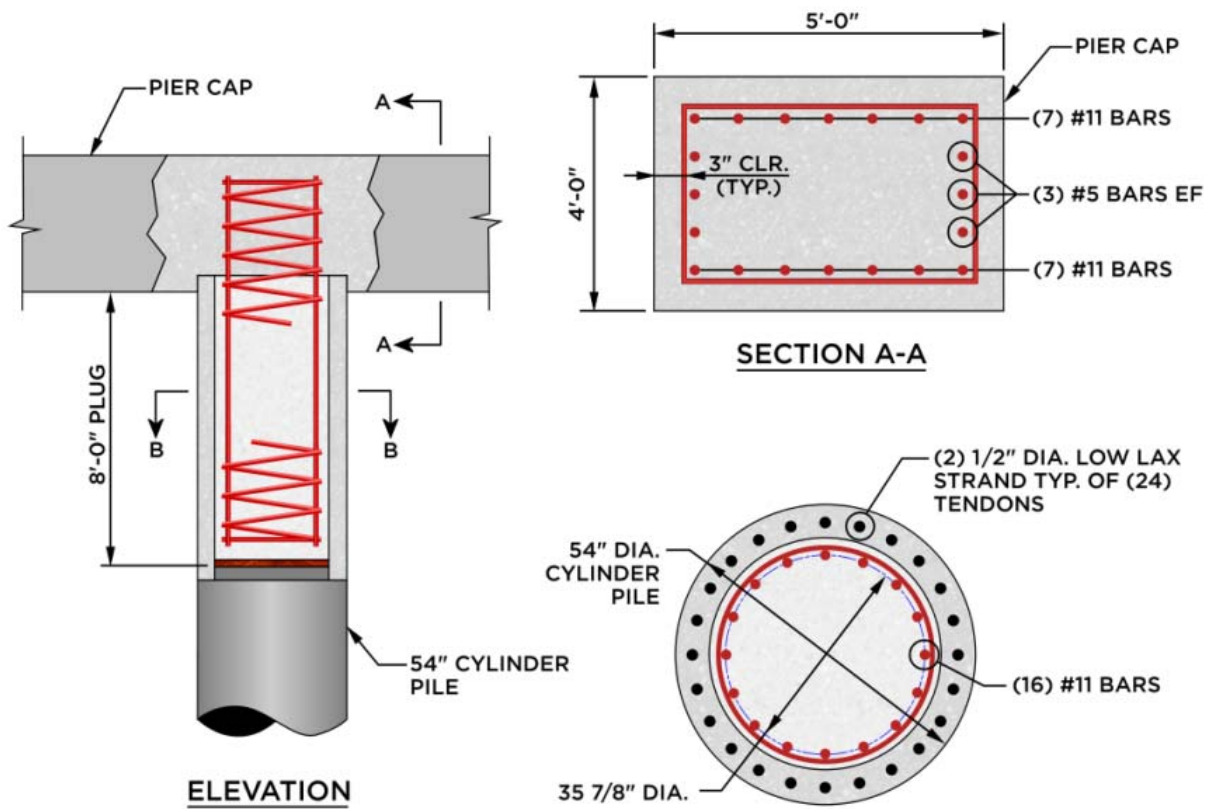


Figure 18: Details of a C.I.P. Pier Cap. The Pile to Cap Connection Utilizes (16) #11 Bars Uniformly Distributed on a Bolt Circle of 35 7/8 inches. The Cap Design is simple, consisting of (7) #11's T & B, (3) #5 Fascia Bars EF, and #6 Stirrups @ 12 inches on center.

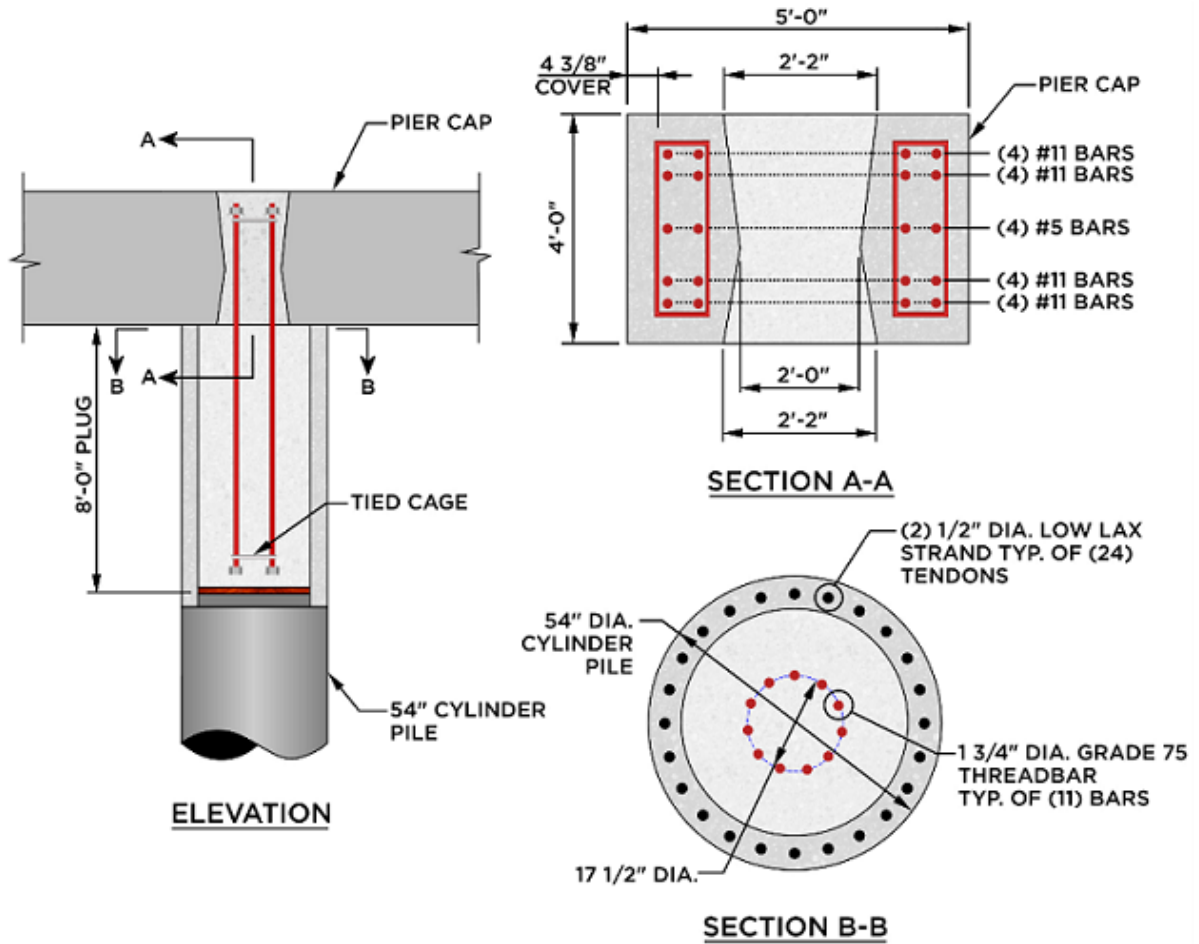


Figure 19: A Precast Alternate to the Pier Cap and Pile to Cap Connection Shown in Figure 18. The Pile to Cap Connection Utilizes (11) 1 3/4" inch Diameter Grade 75 Threadbar, and the Cap Incorporates Hour Glass Voids to Accommodate the Threadbar Cage.

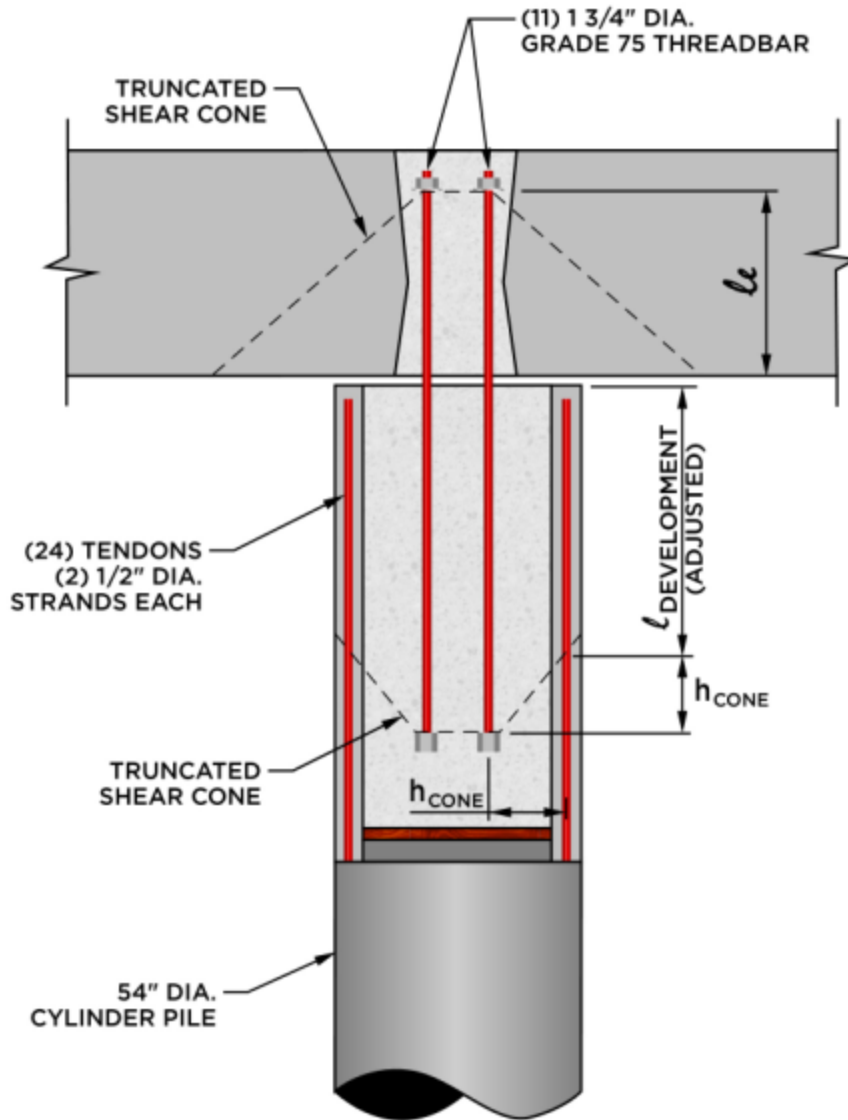


Figure 20: A Schematic of the Pile to Cap Connection Indicating the Development of the Connection in the Pier Cap and Cylinder Pile. The Design Approach outlined in the Paper is Applicable to a Wide Range of Connections. First, the Interaction Diagram for the Section at the Pile to Cap Interface is Obtained and Compared to Factored Design Loads or to the Alternate C.I.P. Connection Interaction Diagram. Second, the Bar Cage Development is Established for the Connection in the Cap and Pile.