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16. Abstract

For the Texas Department of Transportation (TxDOT), the costs and negative impacts associated with bridge construction traffic control and disrupted traffic flow have increased significantly in recent years — particularly in Texas' urban areas. Concerns about worker and driver safety have also heightened as traffic volumes increase. Moreover, bridge construction on large projects and over-water crossings has itself proven to be slow and costly, owing primarily to these projects' requirements for large volumes of cast-in-place concrete and for the associated formwork and field operations.

All of these concerns may be addressed in part through the use of precast bent caps in place of the more traditional cast-in-place caps. In this project, reliable details and a design methodology for connecting precast bent caps to cast-in-place columns and precast trestle piles will be developed, enabling designers to confidently design precast bent cap systems for short- and moderate-span bridges.

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DEVELOPMENT OF PRECAST BENT CAP SYSTEMS AND TESTING PROGRAM

by

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Research Report Number 1748-1

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Project No. 0-1748

Design and Detailing of Precast Bent Cap System

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION

Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

July 1998

IMPLEMENTATION RECOMMENDATIONS

This interim report has no implementation recommendations at this time. The final report will contain the implementation plan.

DISCLAIMERS

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation.

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Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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Chapter 1: Introduction

1.1. Background and Significance of Research

The costs and impact associated with traffic control and disrupted traffic flow have increased significantly over recent years as the Texas Department of Transportation (TxDOT) continues to build bridges in increasingly congested urban environments. Concerns about worker and vehicle safety have also heightened as traffic volume has increased. Furthermore, bridge construction on large projects and over water crossings has proven to be slow and costly due mainly to the large volume of cast-in-place concrete and associated formwork and field operations.

All of these concerns may be addressed in part through the use of precast bent caps, rather than traditional cast-in-place caps. Controlled conditions at precast plants enable the efficient production of large numbers of high quality caps and facilitate the use of high-performance concrete. Application of precast concrete also provides opportunities for reducing construction time and minimizing traffic disruption and the environmental impacts associated with highway construction. Special construction conditions such as sites with difficult access and harsh environments are also more easily accommodated by employing precast concrete.

However, the use of precast concrete for bent caps also presents several challenges. Tighter fabrication and construction tolerances must be met to prevent constructability problems. New construction techniques have to be mastered. Joint details used to connect the precast cap with cast-in-place columns or trestle piles may increase the potential for long-term durability problems, particularly in coastal regions and regions where deicing salts are used. Additionally, uncertainty exists regarding the ability of precast connections to transfer load, especially with the use of inverted tee bent caps, which may require transfer of large forces.

To address these concerns, the Texas Department of Transportation, through the Center for Transportation Research (CTR) at the University of Texas at Austin, initiated Project 1748, "Design and Detailing of a Precast Bent Cap System." Through this project, reliable details and a design methodology for connecting precast bent caps to cast-in-place columns and precast trestle piles will be developed, enabling designers to confidently design precast bent cap systems for short and moderate span bridges. This will help ensure precast bent caps are not only a structurally sound alternative, but also one that is cost competitive with traditional cast-in-place bent caps. Because project results may become a stepping stone to further development of precast substructures, an outgrowth of this research may be the development of new substructure systems that are entirely precast.

1.2. Objectives

To develop standardized details and a design methodology for precast bent cap systems, the objectives of the project are to:

1. Identify new and existing products for anchorage and coupling of reinforcement.
2. Develop practical, cost-effective candidate details for connecting precast bent caps to cast-in-place columns and trestle piles.
3. Test candidate connection details to examine connection constructability, durability and behavior under loading.
4. Provide specific recommendations for standardized details and a design methodology for connecting precast bent caps to cast-in-place columns and trestle piles.

TASK	FY 1997				FY 1998				FY 1999			
	Q1	**Q2**	**Q3**	**Q4**	Q1	**Q2**	**Q3**	**Q4**	Q1	**Q2**	**Q3**	**Q4**
Survey existing literature	■	■										
Identify products for anchorage and coupling	■	■										
Study existing bridges with precast bent caps		■	■									
Develop candidate connection details			■	■	■							
Prepare interim letter report					■							
Test selected details in laboratory						▤	▤	▤	▤	▤	▤	
Implement details in bridge substructure									▤	▤		
Develop final details and design methodology										▤	▤	
Prepare final report											▤	▤

Figure 1.1 Schedule of Research Activities

1.3. Tasks and Schedule

To satisfy the objectives, the following tasks are required:

1. Survey published literature to identify previous studies that have investigated precast substructures and connections.
2. Identify new and existing products for anchorage and coupling of reinforcement.
3. Review precast bent cap connection details used in previous TxDOT projects.
4. Develop candidate details for connecting precast bent caps to cast-in-place columns or trestle piles, in close coordination with TxDOT engineers and an Industry Review Committee (IRC) composed of representatives of the precast and construction industries.
5. Prepare an interim letter report.
6. Test select connection components and "large-scale" connection specimens in the laboratory to examine constructability, durability and behavior under loading.
7. If possible, implement developed connection details in a precast bent cap system to observe the constructability and field performance of the system.
8. Develop final standardized details and a design methodology.
9. Prepare the final report.

The project schedule shows these tasks on a timeline in Figure 1.1.

1.4. Summary of Progress

1.4.1. Literature Survey

As shown in Figure 1.1, progress to date has included work on the first five tasks. The first task included a literature survey of precast substructures and the associated connections. A number of noteworthy projects in recent decades have used precast substructures, such as the Linn Cove Viaduct in North Carolina (1983) and the Chesapeake and Delaware Canal Bridge in Delaware (1995). However, most of the emphasis on connections pertains to the use of post-tensioning schemes developed for project-specific applications, rarely involving bent caps. Limited information is available in the published literature concerning precast substructures, and even less concerning precast bent cap-to-column connections. The vast majority of literature on precast connections centers on building construction in seismic regions, which has restricted applicability to this project.

One notable exception in the literature is a two-year effort currently underway by the Florida Department of Transportation (FDOT) and its primary contractor, LoBouno, Armstrong, and Associates (LAA), to develop a family of standardized shapes and connection details for precast substructures. FDOT plans to use precast components in routine, moderate span bridges for both water and grade crossings. Based on nationwide DOT and industry surveys and on evaluation criteria, FDOT investigated various candidate bent cap and column configurations for hammerhead and multi-column bent applications. Figure 1.2 shows the cross sections for solid rectangular (IA) and inverted-U (III) bent caps that were selected from various candidates for further development. These caps would be connected to hollow rectangular (IIIB) or H-shaped (IVA) precast columns, as shown in Figure 1.3. In most cases, the weight of elements would be limited to approximately 120 kips for handling purposes. Among the various options considered for connections were mechanical couplers, post-tensioning, welded connections, and grout pockets. Preliminary connection details have focused on grouted sleeve couplers and large-volume concrete fills. An industry review board has evaluated the developed details, and completion of standard drawings and design examples is scheduled for early 1998. No testing of details is planned.

1.4.2. Products for Anchorage and Coupling

Various new and existing products for anchorage of reinforcement and coupling of bent caps and columns have been identified. Available products include headed reinforcement, mechanical couplers, and grouted sleeve couplers. To maximize the number of options, proprietary products will not be specified in the details. The following figures show some of the many products that are available on the market. These, of course, should not be interpreted as “recommended.”

Figure 1.4 shows Headed Reinforcement Corporation’s *T-headed Bar* and the Lenton *Terminator* as two typical examples of effective, readily available anchorage devices for reinforcement. HRC has also recently developed an “upset” headed bar which uses a head smaller than the *T-headed Bar*, thereby accommodating smaller tolerances. Devices such as these are useful for the grout pocket alternatives discussed later in Chapter 2. In addition, headed reinforcement may be required where there is insufficient space for development of straight bars or congestion related to the use of hooked bars. Reinforcement may be standard reinforcing bars although cases of large moment transfer may require high strength bolts, such as Dywidag bars or Williams all-thread.

Mechanical couplers are commonly available, including the Richmond and Cadweld couplers. Among grouted sleeve couplers, those produced by NMB and Erico have been used in bridge construction. Figure 1.5 shows the Erico Lenton *Interlok* as one example. Tolerance requirements and other issues associated with grouted sleeve couplers are discussed in Chapter 2.

1.4.3. Study of Existing Bridges with Precast Bent Caps

1.4.3.1. Red Fish Bay

The third task consisted of a study of existing bridges with precast bent caps. In recent years, TxDOT has constructed two bridge projects using precast bent caps: 1) Red Fish Bay and Morris & Cummings Cut Bridges along the Gulf Intracoastal Waterway on State Highway 361 between Aransas Pass and Port Aransas (1995), and 2) Pierce Street Elevated Freeway on Interstate Highway 45 in downtown Houston (1997). TxDOT also developed an alternate design, the Burlington Northern Bridge, for the Texas State Railroad using a precast bent cap system. FDOT has similarly incorporated precast substructures on selected projects, including the Edison Bridge over the Caloosahatchee River in Fort Meyers. A brief summary of each precast system and lessons learned during construction are described in this section.

The Red Fish Bay and Morris & Cummings Cut Bridges (abbreviated as Red Fish Bay) were designed and constructed as replacement structures because of severe salt-water induced deterioration in the original bridges. Red Fish Bay incorporates a pretensioned double tee superstructure supported by precast rectangular bent caps on precast trestle piles. The original substructure design called for cast-in-place bent caps on precast piles. However, the contractor requested to use precast caps to minimize concrete operations over water. With slight modification, a contractor-proposed detail for connecting the precast cap to piles was implemented.

The construction sequence first required driving piles using a steel frame as a template to satisfy tight tolerances. After piles were driven and cut back to the proper elevation, two #9 U-shaped epoxy-coated dowel bars were epoxy grouted into embedded sleeves which had been cast in each pile top. As shown in Figures 1.6

and 1.7, the precast cap was then lowered over the dowels and set on the piles. Due to slippage of the shims, friction collars at exterior piles were used to temporarily support the cap. After adjusting cap elevations and forming around the voids, concrete was cast in the slotted voids to complete the connection (Figure 1.8).

Several lessons were learned through this highly successful project. Use of precast caps (more than sixty) greatly expedited construction operations, enabling construction to finish six months ahead of schedule. Durability provisions included epoxy-coated bars, a non-shrink concrete with inorganic corrosion inhibitor, and a precast substructure and prestressed superstructure. These measures are expected to increase the structure life about 50% at an initial cost increase of only 5%.

Improvements may be made as well. There still remains some concern over potential durability problems related to shrinkage in the grout pockets. Possible cracking at the pocket top due to grout shrinkage could provide a path for moisture ingress. In addition, alternative connection configurations should be developed to ease tolerance requirements associated with fitting a precast cap over driven piles (see Figure 1.6). Chapter 2 addresses other approaches to support bent caps during erection and identifies other grout pocket configurations suitable for efficient precast operations and force transfer.

1.4.3.2. Pierce Street Elevated Freeway

Similar to Red Fish Bay, the Pierce Street Elevated Freeway (abbreviated as Pierce Street) construction in Houston involved replacement of significant portions of the bridge superstructure and substructure that were damaged primarily by corrosion of reinforcement. Rapid replacement of Pierce Street was even more crucial than Red Fish Bay because of the central location of the busy 1.6-mile structure that connects downtown Houston with Galveston Island and the southeast suburbs. The original Pierce Street design used cast-in-place inverted tee bent caps on cast-in-place columns to support the pretensioned girders and cast-in-place deck. The remarkable success of the Pierce Street project has been largely attributed to TxDOT's decision and the contractor's ability to rapidly replace the existing superstructure and bent caps with precast elements. Existing columns were salvaged by saw-cutting column tops after removal of the cap**.**

To transfer substantial forces between the inverted tee cap and column, a bolted connection was designed and constructed. High strength Dywidag bars were first grouted into drilled holes in column tops, then proof tested. Figure 1.9 demonstrates the procedure developed to "thread" the cap over dowel bars with the aid of small-diameter pipes of varied lengths extending through the cap depth. A 150-ton crane was used to place heavy single-piece precast caps. After the cap was set to the proper elevation using shims, the bedding layer at the cap-to-column interface was formed, and grout was poured through large diameter tubes (Figure 1.10). The ducts encasing the Dywidag bars were then filled with grout and the bars tightened at the cap-top anchorage. Finally, recessed anchorage pockets were grouted.

Pierce Street also demonstrated the crucial advantage of precast elements in expediting construction operations, despite requiring less familiar and more complex operations for connections. However, a number of improvements may be suggested. Durability, especially of the exposed areas of the bedding layer (Figure 1.10), should be investigated because of possible cracking due to transfer of forces related to the large eccentricity of inverted tee caps. Shrinkage and creep of the grout and associated stress concentrations at the shims may also produce cracking that could contribute to durability problems. Lastly, the large number of on-site construction operations associated with completion of the connection suggests the need to simplify the connection.

1.4.3.3. Texas State Railroad Bridge

The precast alternate for the Texas State Railroad Bridges used precast caps that rest on driven steel H-piles and support precast box beams. An isometric view of this system is shown in Figure 1.11. While the construction sequence is similar to the sequence used at Red Fish Bay, the grout pockets and anchorage differ significantly. Instead of using dowels, the stability of the connection and anchorage depend on adequate pile embedment into the caps. For this reason, the piles were embedded into the cap approximately half the cap depth. To facilitate bearing, a steel plate welded to the top of the piles was used. Though not built, this option appears to have similar advantages and concerns as Red Fish Bay.

1.4.3.4. Florida Edison Bridge

The Edison Bridge in Fort Meyers, Florida, was designed to replace the original bridge due to a large accident rate and serious deficiencies in the substructure. Figure 1.12 shows the precast bents used to support the prestressed girders and deck. Inverted-U caps and I-shaped columns were chosen to minimize weight. Proprietary grouted-sleeve couplers connected the cap and column, and templates were used to meet tight connection tolerances at both the top and bottom of precast columns. In addition, cap tops were horizontal to ensure caps were successfully placed over dowels. A cross slope of two percent was achieved by varying pedestal heights. The successful use of precast caps on the Edison Bridge indicates that grouted-sleeve couplers may be another feasible option for connections. In this case, the speed of erection and cost depend on the ability of contractors to satisfy tight tolerances. This approach also requires the use of proprietary grouted sleeve couplers, which preclude the use of alternate connection hardware and may increase cost.

1.4.4. Development of Connection Details and Test Program

Specific guidance was given by TxDOT personnel in the early stages of the project to limit the number of candidate connections to the most common cases. These included bent caps of rectangular and inverted tee cross sections supported on either cast-in-place columns or precast trestle piles. Depending on superstructure span lengths and pier spacing for these cases, connections at cast-in-place columns may be required to transfer either large or small moments that are a function of the loading. Large moment transfer would commonly be expected for unbalanced back and forward superstructure spans used in combination with inverted tee caps. In contrast, the shorter spans used with closely-spaced trestle piles would normally result in a low moment requirement. Table 1.1 summarizes these cases. Connection forces including moment, axial force, and shear for a range of bents is provided in chapter two, together with implications for design and testing.

Table 1.1 Matrix of Connection Applications

	Trestle Piles		Cast-in-place Columns	
Cap Type	**Rectangular**	**Inverted Tee**	***Rectangular***	***Inverted Tee***
Low Moment	Common	Rare	Common	Common
High Moment	N/A	N/A	Common	Common

The following two chapters summarize progress in the development of connection details and outline the scope of the laboratory test program. Chapter 2 presents the connection details that have been developed and the rationale for further development or elimination of specific details. Chapter 3 outlines a preliminary three-phase test plan.

Chapter 1 Figures

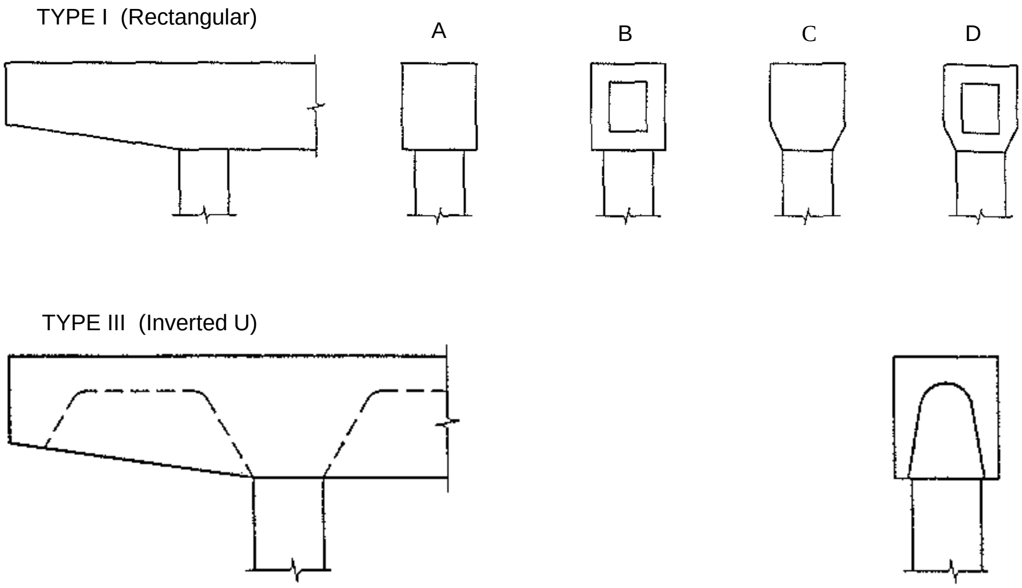


Figure 1.2 Elevation and Section Views of FDOT Standard Shapes for Precast Bent Caps

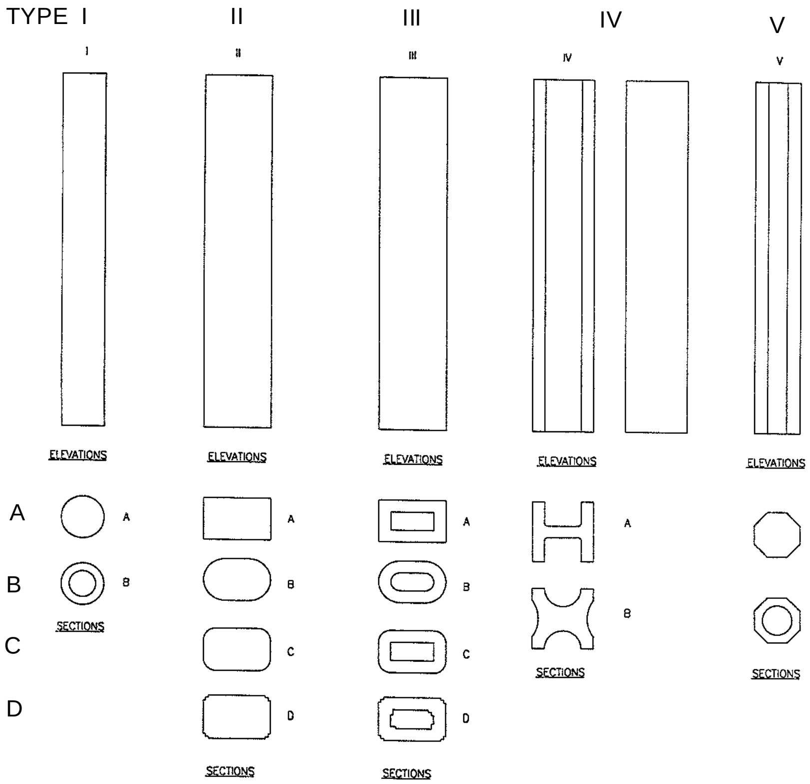
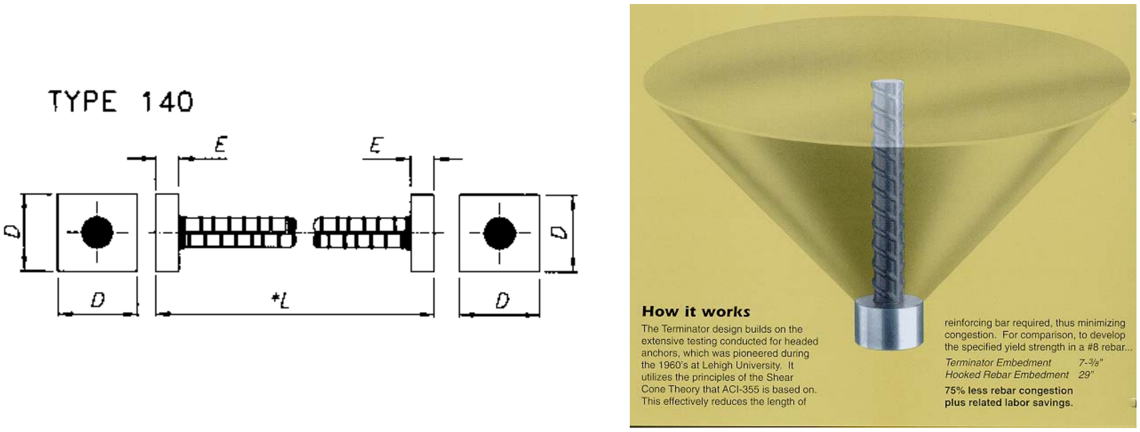


Figure 1.3 Elevation and Section Views of FDOT Standard Shapes for Precast Columns



How it works

The Terminator design builds on the extensive testing conducted for headed anchors, which was pioneered during the 1960's at Lehigh University. It utilizes the principles of the Shear Cone Theory that ACI-355 is based on. This effectively reduces the length of reinforcing bar required, thus minimizing congestion. For comparison, to develop the specified yield strength in a #8 rebar...

Terminator Embedment	*7-3/8"*
Hooked Rebar Embedment	*29"*

75% less rebar congestion plus related labor savings.

Figure 1.4 HRC's T-headed Bar and the Lenton Terminator

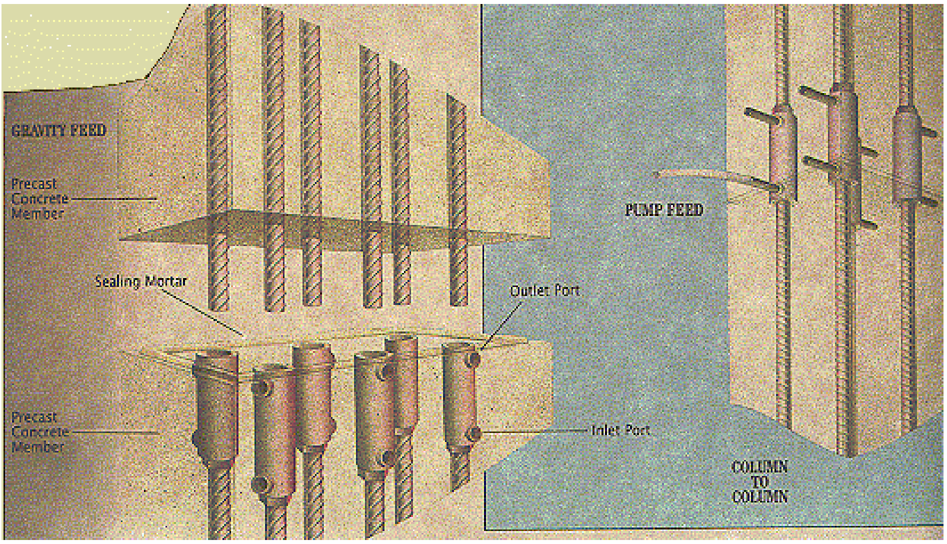


Figure 1.5 The Lenton Interlok Grouted Sleeve Coupler

Figure 1.6 Lowering Precast Cap over Dowels onto Piles

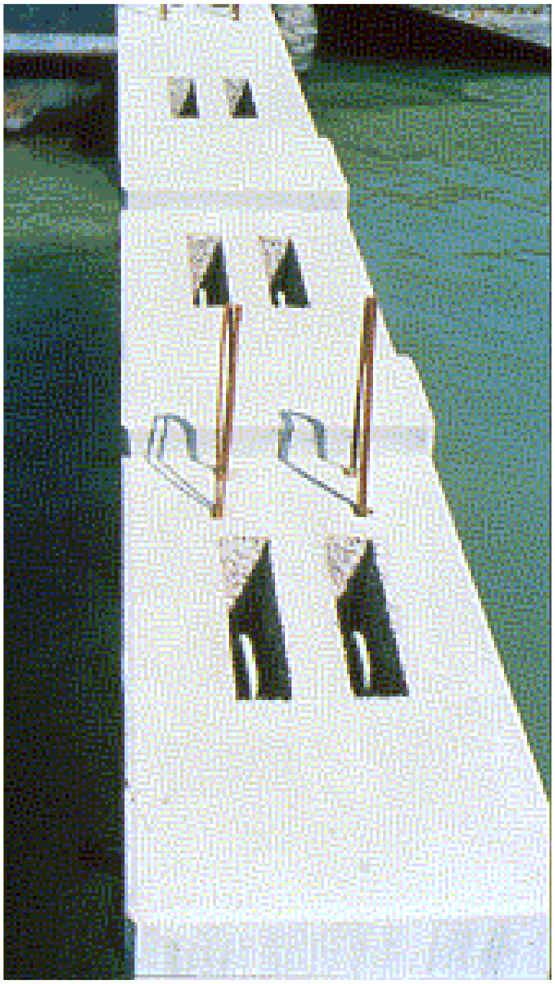


Figure 1.7 Grout Pockets

Figure 1.8 Workers Pouring Concrete into Slotted Voids

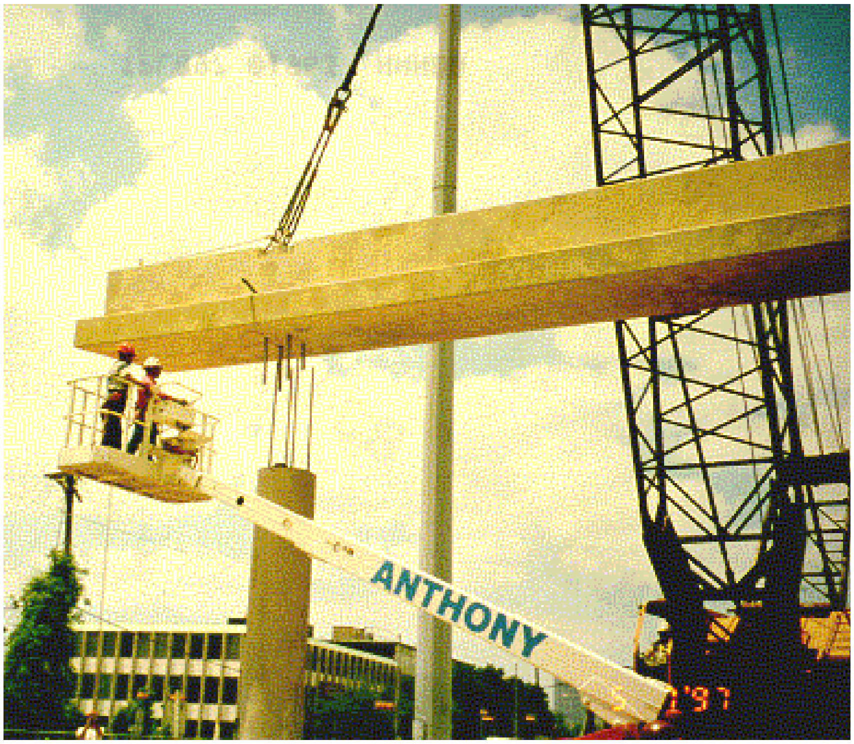


Figure 1.9 Workers Being Positioned to Help Thread Dowel Bars through Precast Bent Caps with Aid of Extended Pipes

Figure 1.10 Precast Bent Cap Resting on Shims prior to Grouting of Bedding Layer and Sheathing Ducts

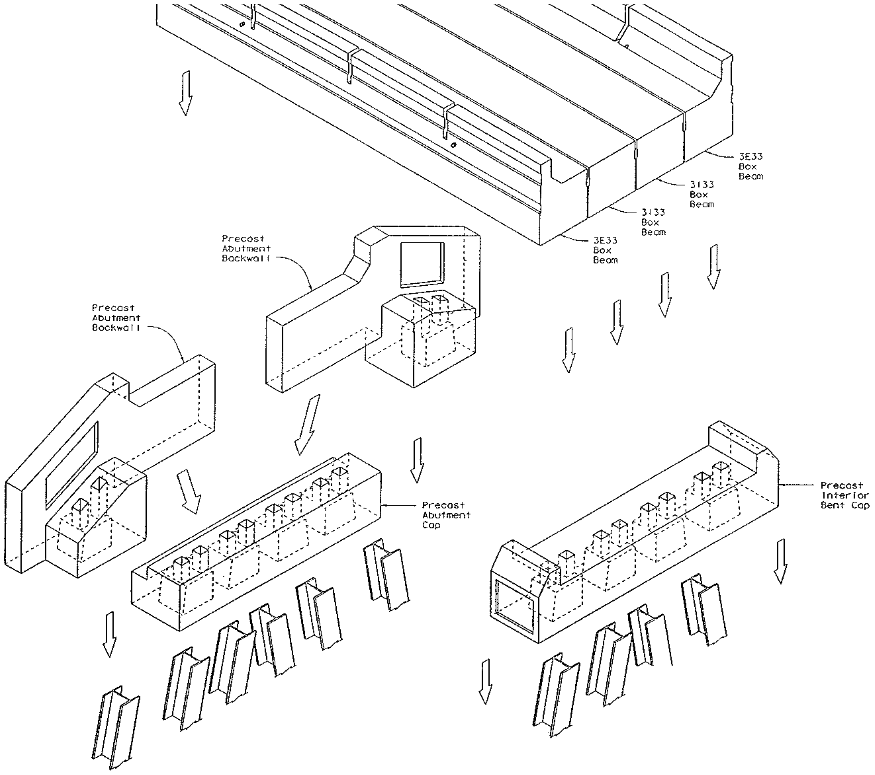


Figure 1.11 Isometric of Texas Railroad Precast System

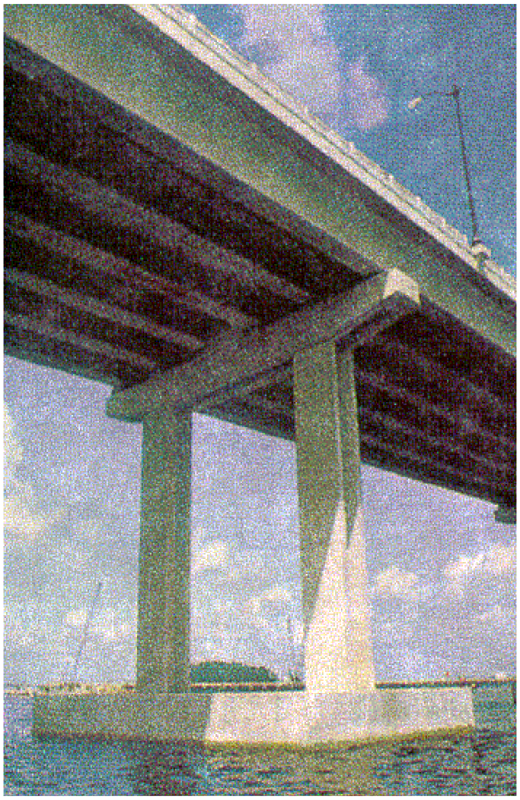


Figure 1.12 Typical Precast Bent on the Edison Bridge

Chapter 2: Development of Connection Details

2.1. Introduction

2.1.1. Industry Review Committee

Based on a review of existing literature, available connection hardware, and existing bridges using precast bent caps, various candidate connection details were developed. It was recognized from the project inception that The University of Texas research team should develop connection details in close coordination with both TxDOT engineers and representatives of the precast and construction industries to ensure details are practical and cost-effective. An Industry Review Committee (IRC) consisting of the following six industry representatives was formed: Charlie Burnett (Champagne-Webber, Houston); Paul Guthrie (Texas Concrete, Victoria); Fred Heldenfels IV (Heldenfels Enterprises, Inc.); Randy Rogers (McCarthy Brothers, Austin), Chairman; Carl Thompson (Dalworth Concrete); and Roger Welsch (Association of General Contractors, Austin). Constructive comments provided through two review cycles were instrumental in the development of connection details. Issues of economics, constructability, durability, and force transfer were addressed during the reviews. These issues are briefly summarized in the next section. Details for the three connection types–grout pockets, grouted sleeve couplers, and bolted connections–are then presented, together with a summary of the primary advantages and disadvantages of each detail. Reasons for further development or elimination of a detail are also provided.

2.1.2. Issues

2.1.2.1. Economics

The development of reliable precast bent cap systems is highly motivated by the potential economic advantage of precast construction, including reduced construction time. Precast connections must be developed as integral components of competitive precast alternatives to traditional cast-in-place construction. From the perspective of both initial and life-cycle costs, the economic benefit of a precast bent cap system is primarily related to constructability and durability, and to some extent force transfer. Issues such as connection types and configurations, selection of materials, and construction operations must be investigated together with provisions for durability and structural efficiency.

2.1.2.2. Constructability

To maximize construction efficiency and cost-effectiveness, connections must be easy to build. Constructability issues apply to both fabrication and erection. Development of the following connections has included consideration of issues such as formwork, grout pocket configuration, bedding layer thickness, tolerances, type of anchorage, and specification of non-proprietary grout and bolts. Construction complexities such as match-casting and post-tensioning have been intentionally minimized because they are thought to detract from the cost-competitiveness of this class of precast construction.

2.1.2.3. Durability

Durability is a major concern for bridge substructures in Texas. State-owned bridges that have been rated as deficient typically have substructures with advanced concrete deterioration, often due to insufficient cover resulting in corrosion of reinforcement and spalling. Figure 2.1 shows corrosion damage of the original Pierce Street bridge that precipitated replacement of its superstructure and bent caps. Cracks and other moisture paths into connections may facilitate moisture ingress, resulting in corrosion of connection reinforcement. Regions of particular concern include exposed surfaces of grout pockets or ducts, the bedding layer between the precast cap and columns or trestle piles, and bolted connection anchorage zones. Cracking may be introduced in a number of ways, such as shrinkage in grout pockets or transfer of forces at service level. Durability must also be addressed through specification of grout properties. For economy, a cementitious grout is desirable. However, to address durability concerns grout should exhibit minimal shrinkage yet provide sufficient resistance to cracking, low permeability, flowability, and a compressive strength at least equal to that of the precast components. Provisions such as large cover and epoxy-coated anchorage reinforcement are also viable. Further precautions such as external sealants, epoxy grout, latex-modified concrete, post-tensioning, and even waterstops may be advisable for certain applications.

2.1.2.4. Force Transfer

Precast Connection Issues

The development of precast connections to simulate monolithic behavior requires that a design methodology be established to account for transfer of forces. To address the various uncertainties in behavior and to accurately “calibrate” design provisions to actual behavior, a test program will be conducted to experimentally verify load paths, failure mechanisms and capacities. Designers will then be able to determine by analysis the required capacity for a given connection and provide sufficient strength in design, based in part on experimental results. With a sound design methodology, maximum efficiency in design is possible.

Because precast connections are the critical components in the load path between the superstructure and substructure, all forces on these connections must be taken into account. Accordingly, the usual assumptions for design of cast-in-place bents must be revisited to determine major differences for design of precast connections. The test program will help resolve some of the following issues related to connection design.

1. *Connection Design Forces.* From experience, standard practice for continuity of column bars and bar projection into cast-in-place caps has proven adequate. Because a more limited number of connectors would be used for a precast connection, connector capacity must be verified (or at least shown to be equivalent to standard practice). While simplicity is a goal of the design methodology, the approach must still, in some manner, account for the connection demand due to simultaneous transverse (in-plane) moments and longitudinal (out-of-plane) moments on the bent, as well as the corresponding axial force and shears. Significant longitudinal moments are anticipated due to unbalanced live load, especially for inverted tee caps. Future use of the AASHTO LRFD Specifications will substantially increase longitudinal design moments because of the more severe live load and braking requirements. Although load combinations for dead and live load often govern the design of caps, moments due to lateral loads should also be investigated in load combinations when investigating transfer of moment through a connection (i.e., column top moments).
2. *Capacity of Bars*. Capacity of individual bars and bar groups must be established for design. Connector arrangements may typically include one or two connector lines with two to four connectors per line. Appropriate failure modes such as bar yield, cone pullout, and side blowout must be accounted for. As described in Chapter 3, the test program will investigate anchorage capacity as well as determine connection response under realistic loading.
3. *Reinforcement in the Joint Region*. Design of moment-resisting connections for cast-in-place construction normally accounts for joint confinement, joint shear, and development of beam (and sometimes column) bars in the joint. Precast connections should satisfy similar provisions.

Analyses for Connection Forces

A series of seven analyses were performed to bound the range of connector forces for multi-column cast-in-place and trestle-pile bents with rectangular and inverted tee caps. Table 2.1 summarizes the bent configuration for each analysis. Results of these analyses provide insight into design requirements and furnish important guidance for testing. Testing will help ensure development of a design methodology that accurately accounts for anchorage capacity of connectors.

Figure 2.2 shows a bent with an inverted tee cap located near the Mopac (Loop 1) overpass at Town Lake in Austin. This configuration, shown idealized in Figure 2.3, provides an upper bound on design moments. It was used as the basis for the IC1 analysis summarized in Table 2.1. Column heights were assumed in the range of 12’ to 18’ (clear). Two cases of adjacent span lengths were used with AASHTO Type IV girders: 1) equal 100’ spans, and 2) unequal 115’ and 85’ spans. Transverse moments were calculated using a 2-D frame analysis, and for comparison, load combinations specified by both the 1996 AASHTO Standard Specifications and the 1994 AASHTO LRFD specifications were checked. Patterned live loads on one span adjacent to the cap produced maximum longitudinal moments. Analyses were performed to determine the maximum moment in each direction, as well as the simultaneous moment in the orthogonal direction.

Table 2.1 **Bent Dimensions Used in Analyses**

Case	Bent Configuration	Cap Cross Section	Girder Eccentricity
RC0	Two 3.5’ Columns at 23’ 30’ Cap	Rectangular 42”x48”	10”
RC1	Two 3.5’ Columns at 20’ 30’ Cap	Rectangular 42”x48”	10”
RC2	Three 2.5’ Columns at 21’ 54’ Cap	Rectangular 34”x42”	10”
RC3	Four 3’ Columns at 16’ 60’ Cap	Rectangular 42”x48”	10”
IC1	Two 3.5’ Columns at 33’ 55’ Cap	Inverted Tee 84”x24” flange, 42”x54” web	28”
IC2	Three 3.5’ Columns at 34’ 92’ Cap	Inverted Tee 75”x18” flange, 42”x54” web	28”
RP1	Four 1.5’ Piles at 8’ 30’ Cap	Rectangular 24”x24”	8”

Results of analyses demonstrate the importance of accounting for both simultaneous moments in the longitudinal and transverse directions, as well as differences in response for rectangular and inverted tee caps. Figure 2.4 shows example connection forces for RC3 and IC2, respectively, using LRFD load combinations and a column height of 12’. Cases for equal adjacent spans (RC3 and IC2) and unequal spans (RC3U and IC2U) are shown, with actions corresponding to the maximum moments in each direction. Figure 2.4a shows the relatively small connection forces and eccentricities (moment/axial force) typical of multi-column bents with rectangular caps, whereas Figure 2.4b shows the much larger connection forces and eccentricities characteristic of inverted tee caps. Additional calculations showed that for cast-in-place columns with rectangular caps two lines of #9 connectors with two connectors per line are expected to provide adequate transfer of connection forces. Inverted tee caps typically would require two lines of #11 connectors with four connectors per line, and piles would require only a single line of two or three #9 connectors. Figure 2.5 shows a plan view of connectors for these typical cases.

2.2. Development of Candidate Connection Details

This section presents the features of the connection details in three categories: grout pockets, grouted sleeve couplers, and bolted connections. For each category, original “draft” details are presented in plan and section views along with the construction sequence. Then, revised connection details are shown, based on committee comments. Rationale is also given for eliminated and added details.

2.2.1. Grout Pockets

The use of grout pocket connections has several advantages:

1. Fairly simple and familiar construction practices that have been used for years to accommodate construction tolerances and provide “forgiveness” of construction error;
2. Pocket shapes that may be tailored to satisfy constructability and force transfer requirements;
3. Behavior that is understood;
4. Successful use in building construction and, to a limited extent, in bridge construction.

For these reasons, grout pocket options have been developed for rectangular and inverted tee bent caps for use with either precast trestle piles or cast-in-place columns. The challenges associated with using grout pockets are addressed in the following sections.

2.2.1.1. Grout Pocket 1 and Refinements

Figures 2.6 and 2.7 show the details for Grout Pocket 1, GP1, and Grout Pocket 2, GP2, respectively. GP1 is a variation of the Red Fish Bay detail, and GP2 is essentially the Red Fish Bay detail with a single tapered grout pocket. Although the details show caps connected to driven piles, similar details may be used with cast-in-place columns. The construction sequences, described earlier for Red Fish Bay (section 1.4.3.1), are shown also with the figures. The two primary differences between GP1 and GP2 are the type of connection reinforcement and the location of the pile top relative to the bottom of the cap. GP1 uses headed reinforcement instead of the hooked bars shown for GP2. Use of headed bars provides a greater number of connectors for a given pocket length, as well as the possibility of a single line of connectors. In addition, with GP1 the top of the pile remains completely beneath the cap and has a flat bedding layer to accommodate tolerances. In contrast, GP2 requires that the cap actually enclose the pile top. Although GP2 ensures proper alignment of piles relative to the cap, it requires tighter tolerances to be met. Committee comments related to constructability, durability and force transfer are summarized in Table 2.2.

Table 2.2 **Committee Review Comments for GP1**

Factor	Comments
Constructability	**Steps of construction sequence**: Step 1) template not common; Step 3) use shims, not collars. Also, use grout bearing seat build-up to correct for elevation error (~1.25”).
	Difficulty in grouting bedding layer and preventing air pockets: Verify grout with vent tubes. Provide at least one inch of vertical tolerance at bedding layer. Consider “viscosity” standard for grout.
	Installation of headed reinforcement: Epoxy grouting of headed rebar should be a straightforward operation. Either precast or drilled holes may be used. Verification of (threaded bar) anchorage in field is possible. Full depth pockets ensure vertical tolerance is not a problem and are easier to build and inspect. Epoxy-coated bars may be slightly more expensive.
	Grouting operation: Minimize grout operations. Connection grouting and pocket grouting will need to be separate operations to ensure quality control. Connection grouting will probably use epoxy, which gains strength very quickly and is best suited for small volumes because of cost and material properties. Cementitious grout would likely be used for the larger-volume pockets.
	Shim stacks vs. friction collars: Because of limited capacity of friction collars, shims should be accounted for in the design. Research must determine the extent and impact of load concentration at shims. Less rigid, polypropylene shims may be better than steel.
	Potential for crooked piles: More stringent tolerances and the use of a pile-driving template (as for Red Fish Bay) will help. However, tolerances beyond the norm for cast-in-place add cost and damage the economics of the precast option.
Durability	**Cracking at bedding layer producing path for moisture ingress**: Extent of cracking should be investigated through testing at service level loads. This is considered better than the potential cracking in GP2, which could not be observed and repaired.
	Top of grout pocket exposed: Could waterproof. May not be a problem with large depth to rebar.
Force transfer	**Low redundancy and moment capacity for single line of connectors**: Either use at least two rows *or* adjust load factors.
	Grout pocket requires verification of anchorage and connection strength: Consider roughened, beveled or stay-in-place form. Could invert form for anchorage. Keep top wide to cast grout.
	Stirrups: Verify requirement. Difficult to place. Use large reinforcing bars to avoid congestion.
Other	**Pile sizes**: Piles are commonly 16” or 18” square, with a small percentage of 20” and other sizes.

Based on these comments, revised details, GP1A and GP1B, were developed, as shown in Figures 2.8 and 2.9, respectively. Primary changes were related to the grout pocket configuration: GP1A uses a roughened double bevel to enhance anchorage of hooked bars and simplify form removal, while GP1B uses a roughened “inverted taper” and allows a stay-in-place form to enhance anchorage in a small pocket with headed reinforcement. Refinements such as the use of epoxy-coated headed or hooked bars, non-shrink grout, and a bleed port were applied to GP1, as well as all subsequent grout pocket details. Shims were also considered an alternative to friction collars. Additional refinements to GP1 included the use of 18” square piles and elimination of the stirrups used in Red Fish Bay. Requirements for joint confinement reinforcement will be determined through testing.

Comments made during the second committee review suggested further changes. Because of the difficulty in form removal with a roughened interface, the use of a more cost-effective, reusable (smooth-faced) steel form was recommended. An alternative using a single inverted taper through the depth of the pocket was highly recommended because of the advantage of form removal from one side. This option is shown in Figure 2.10 (GP1C) with a pocket taper similar to GP1 but inverted. Various angles of taper inclination are possible. Angles as large as approximately five degrees are possible for pile caps using a single line of connectors, while the taper must be restricted to much smaller angles for cast-in-place caps with double connector lines to ensure the pocket fits reasonably within the cross section. Another option is to minimize the pocket size through the cross section by using a nearly vertical pocket with minimal form draft. Connector anchorage could be provided by tapering the pocket along its length, rather than through the cross section. The volume of the pocket should be kept as small as possible to minimize congestion problems and interference with cap reinforcement, to simplify grouting operations, and to limit shrinkage.

Durability issues were also addressed. Provisions such as a waterstop joint or external sealant were thought to add undue complexity. However, application of a waterproof agent may be viable, if needed. It was recommended that the requirement for sealants be determined by observing the extent of cracking in specimens under service-level loads during testing. Recent inspection of cracking at the top of select grout pockets at Red Fish Bay revealed very sparse, small cracks (~0.005 in.). The grout mix, which will be an “extended” grout or small-aggregate concrete, should also be designed for durability, accounting for shrinkage, permeability, and flowability.

2.2.1.2. Grout Pockets 2 and 2A

Although many committee comments for GP1 similarly applied to GP2, there were a few notable differences. The requirement that the cap in GP2 enclose the pile top caused considerable concern for three reasons: 1) more extensive field operations may be required to achieve the tight horizontal tolerances; 2) based on previous Red Fish Bay experience, use of shims is not feasible (although friction collars are a viable option); and 3) cracking and deterioration of the connection cannot be monitored or easily repaired. While the detail appears to provide a more difficult path for moisture ingress with adequate grouting, capillary action may nullify this apparent advantage. Because these liabilities seemed to outweigh advantages from prior use of GP2, the committee decided to discontinue further development of this detail. Nevertheless, such a detail may be useful for cases involving battered piles. The GP2A detail shown in Figure 2.11 combined the features of GP1 and GP2, but was not developed further for the same reasons as GP2.

2.2.1.3. Grout Pocket 3 and Refinements

Application of grout pockets to inverted tee caps generates several additional issues due to the larger size cap and different amounts and locations of reinforcement. Figure 2.12 shows the original GP3 connection, including the shallow “inverted cone” pockets with headed reinforcement and horizontal ducts for pressure grouting. In addition to comments similar to those for GP1, committee comments for GP3 centered on the size and orientation of the grout pocket, as well as grouting operations. While it was recognized that inverted cone pockets should provide better anchorage, they would require stay-in-place forms. In addition, pockets could be changed in width and height to minimize grouting operations and facilitate the use of additional headed or hooked bars. Incorporation of a sloping duct from the stem sidewall was also recommended to facilitate gravity-flow grouting. However, adequate grouting of the bedding layer would still need to be verified during testing and field inspections.

Based on committee feedback, GP3A was developed, as shown in Figure 2.13. Taller and wider pockets enable the use of hooked bars or greater numbers of headed bars and also permit gravity-flow grouting. The roughened interface enhances anchorage. The committee pointed out that the pocket height could interfere with the closely spaced inverted tee ledge steel. Shorter pockets, small circular ducts with headed bars (as shown in the plan view of GP3), and greater ledge steel spacing would make possible other alternatives, such as that shown in Figure 2.14. Another concern was the use of a rectangular shaped pocket, which could be costly to form and difficult to grout. Use of pockets with smoothed edges or stay-in-place forms would be effective, though more costly. One recommendation was to use standard metal or PVC post-tensioning ducts with headed anchors. Another suggestion was to extend a pocket the full depth of the cap (Figure 2.25). In this option, the cap could serve as a template to drill holes in the column for anchors, followed by cleaning of the holes and pocket and then grouting of the pocket. This would eliminate tolerance issues associated with the anchorage reinforcement. It should be noted that this last option would also be an alternative for rectangular caps. Testing will address various issues associated with these details.

2.2.1.4. Grout Pocket 4

Figure 2.15 shows GP4, an option with the pocket formed in the column rather than in the cap. Despite recognized deficiencies, GP4 was presented to identify issues that could help in the overall development of grout pocket alternatives. One original concern was the requirement for field installation of connectors. Committee comments reinforced this concern for two reasons—safety and economics. Upon delivery, the cap would require temporary supports and additional handling for safe on-site installation of couplers on the underside of the cap. These extra operations, together with the use of mechanical couplers, would delay erection and be costly. Furthermore, concern over less reliable and less efficient field forming of grout pockets and the need for pressure grouting contributed to the consensus that this option be eliminated.

2.2.1.5. Grout Pocket 5

GP5 is a variation of a Florida DOT precast substructure connection detail. The original Florida DOT detail is shown in Figure 2.16. The detail uses a grout pocket with extended and confined column bars serving as connectors. Concrete is cast through a filling hole in the cap top to complete the connection. This detail could be applied to low-moment connections with cast-in-place columns. The primary changes to the original FDOT detail include a taller pocket with either an inverted cone or cylindrical shape, a roughened interface to enhance anchorage, and a bedding layer to accommodate vertical tolerances. The committee expressed concern over the apparently large pocket volume, placement of the concrete, and additional fabrication and construction operations associated with the anchorage and bent cap reinforcement. The committee suggested this detail not be developed developed further, but that implementation by FDOT should be monitored.

2.2.2. Grouted Sleeve Couplers

Grouted sleeve couplers have a long history of use with precast concrete beam-to-column connections in building construction. However, their use in bridge applications has been more limited. Reasons for use in bridge construction include:

1. Successful previous use in building construction;
2. Development of anchorage and full continuity of column bars, established by testing;
3. Transfer of tensile loads through reinforcement, not concrete;
4. Rapid and fairly simple completion of the connection.

Figure 2.17 shows a grouted sleeve coupler option with a rectangular cap. Committee comments addressed several issues. The tight horizontal construction tolerances of approximately +/-2/3” maximum were considered a major drawback. Accommodating such tolerances could be very challenging, especially with driven piles. This would require special provisions such as the use of a template for alignment of column bars between precast components, exceptional precaution by contractors, and additional coordination between fabricators and contractors. Because grouted sleeve couplers are proprietary products, contractor options would be limited. Also, pressure grouting would require extra equipment and greater quality control, although this may not be a significant factor for larger jobs. Unlike grout pockets, separate grout operations could be required for grouting the coupler and bedding layer. All of these factors would increase cost. However, because of previous successful use, the committee suggested that this detail be left as an option and that implementation by FDOT be observed.

2.2.3. Bolted Connections

Bolted connections possess certain advantages:

1. Simple and familiar construction practices;
2. Resistance of large moments by use of high strength bars;

3. Minimal on-site grouting;
4. Well-understood behavior;
5. Optional post-tensioning to limit durability problems and verify installation.

Bolted connections may be especially useful for inverted tee bent caps because of the large moment resistance afforded by high strength bars and minimal requirements for on-site grouting. Also, in contrast to the grout pocket options for inverted tees, bolted configurations have minimal conflicts with ledge reinforcement. The following bolted connection alternatives are portrayed in bents using inverted tee caps. If needed, such details could be used with rectangular caps.

2.2.3.1. Bolted Connection 1 and Refinements

Figure 2.18 shows the detail for Bolted Connection 1, BC1, which is essentially the detail used in the Pierce Street Elevated project. As mentioned earlier, the construction sequence involves grouting high strength bars into column-top holes, followed by the "threading" of the cap over these bars with the aid of pipes extending through the ducts and beneath the cap. After grouting the bedding and sheathing, the bolts are tightened. Grouting of the cap top recesses completes the connection.

Committee comments focused on constructability issues. A reduction in grouting operations was recommended. This could be achieved by combining grouting of the sheathing and bedding. It was also suggested that the anchorage recess in the cap top be moved to the slab to alleviate congestion of cap top reinforcement. Alternative construction sequences were also discussed. One alternative would be to first place the cap (with its embedded sheathing ducts) on the columns, followed by drilling and "vacuuming" of the anchorage holes in the column, using the cap as a template. It was also recommended that the number of dowel bars be limited as much as possible to avert potential erection difficulties.

Durability was also addressed. Concerns over shrinkage cracking at the cap top recess could be eliminated by relocating anchorage hardware to the slab and waterproofing the anchorage. Service level cracking of the bedding layer could be an issue because of the large eccentricity for inverted tee caps.

Figure 2.19 shows the revised detail (BC1A) that was developed based on committee comments. Principal changes included: 1) a reduction in grout operations by grouting the bedding and sheathing duct through the sheathing duct; 2) an alternative construction sequence specifying placement of the cap, followed by threading of bolts through the sheathing into threaded anchors in the column tops; and 3) cap top recess alternatives, including an embedded insert or the placement of anchorage hardware in the slab, which could reduce grouting to a single operation.

The second committee review provided further suggestions. It was felt that anchorage hardware should definitely be relocated in the slab, for reasons previously mentioned. Use of a foam-filled encasement for the anchorage would have to be carefully specified to provide sufficient durability protection. This would also accommodate relative movement between the slab and anchorage due to thermal expansion. In addition, a reduction in grouting operations was viewed favorably, although provision for an exit port was deemed essential to ensure adequate grouting of the bedding layer. Both proposed construction sequences were considered feasible. However, threading bars into an embedded concrete anchor after cap placement was considered potentially more problematic because little recourse would remain if threading or tolerance problems arose in the field. Another construction alternative was suggested: 1) place resin capsules in the column top holes, 2) set the cap, and 3) thread bars through the cap into the holes to activate the resin. This differs from the alternative construction sequence mentioned for BC1A only in the method of anchorage. A final suggestion was to specify non-proprietary threadbars, although higher strength bars may be more advantageous. Figure 2.20 shows these provisions incorporated into option BC1B.

2.2.3.2. Bolted Connections 2 and 2A

Bolted Connection 2 was an attempt to address the challenge of developing a detail without the use of grout. As shown in Figure 2.21, a main feature of this detail is the use of a neoprene bearing pad at the cap-to-column interface to accommodate tolerances. The elimination of grout would substantially simplify the construction sequence; after epoxy grouting dowel bars and placing the pad, the cap would be threaded over the bars and the bolts tightened to complete the connection.

Concerns over pad deformability, strength, fabrication, and durability arose during the committee review. Despite some flexibility of neoprene pads, it was felt that provisions for tolerances at the cap-to-column interface as well as between columns might not be adequate. In addition, deformation of a pad may cause local spalling of the concrete due to relative movement that would develop between the concrete and pad. Pad strength could easily be addressed by use of laminated pads. However, the need to accommodate super elevation and slope could require expensive, custom-made pads. Alternatively, the cap could be modified, although this would be costly to implement. Durability could be an important consideration for exposed pads, as replacement of a deteriorated pad could be a difficult and costly operation. Finally, grout might still be required for this detail to provide corrosion protection for the bolts in the sheathing.

These shortcomings were considered sufficiently limiting that a completely new bolted detail, BC2A, was developed that incorporated the use of grout. As shown in Figure 2.22, this detail employs leveling nuts together with a bearing plate at each connector for erection, in place of a bearing pad or shims. After the threaded bars are epoxy grouted into the columns, the leveling nuts are easily adjusted to the correct elevations. After the cap is threaded over the bolts and supported on the plates, forming and grouting of the bedding layer and sheathing are performed. As illustrated in the figure, reliable grouting of the bedding would be accomplished by use of a grout tube that extends from the sidewall to the center of the bedding layer, similar to that successfully used in the Pierce Street bent caps.

Committee response was favorable to this new scheme because of guaranteed field adjustability and precision, and the elimination of shims. The committee indicated that this type of detail might be useful for cases requiring large superelevation. Both BC1B and BC2A were considered feasible bolted-connection alternatives.

2.3. Summary of Candidate Connection Details

Based on an evaluation of the advantages and disadvantages of connection details through the review cycles, several details have been selected for further development and testing. These candidate connection details are classified in Table 2.3 according to specific application and are shown in Figures 2.23 through 2.26. The following chapter summarizes a test program to investigate these details.

Table 2.3 **Matrix of Candidate Connections**

	Trestle Piles		**Cast-in-place Columns**	
	Rectangular	***Inverted Tee***	***Rectangular***	***Inverted Tee***
	GP1A GP1B GP1C	GP3A GP3B GP3C	GP1A GP1B GP1C (GS1) (GP5)	GP3A GP3B GP3C BC1B BC2A
High Moment	N/A	N/A	GP1A GP1B GP1C GS1	GP3A GP3B GP3C BC1B BC2A

Chapter 2 Figures

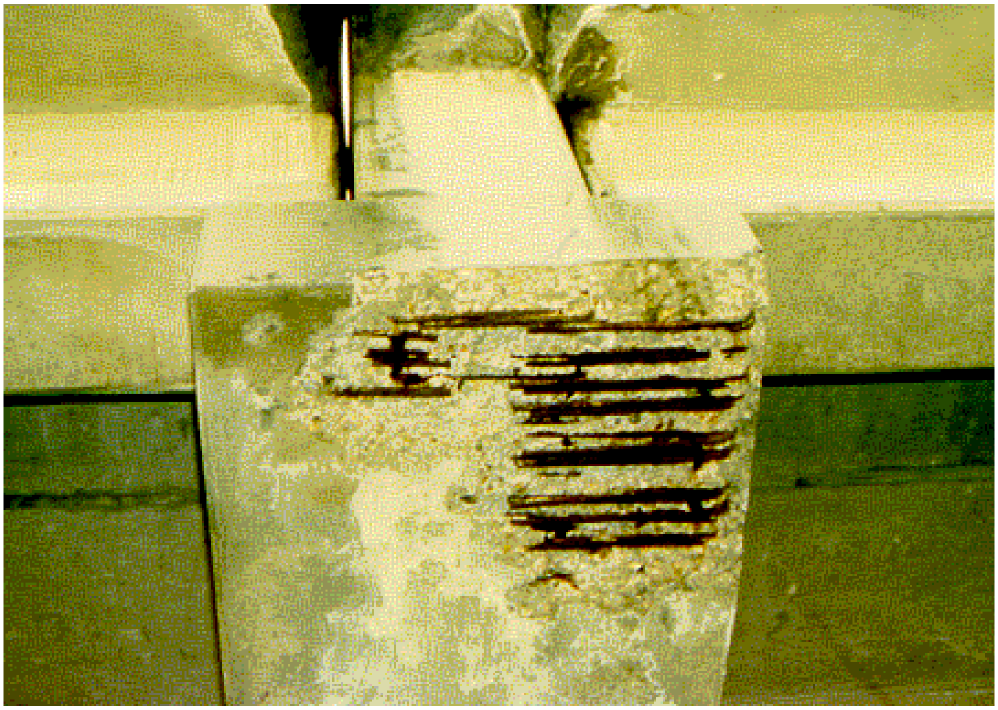


Figure 2.1 Corrosion of Original Pierce Street Bridge

Figure 2.2 Dimensions Used in Inverted Tee Bent Analysis, IC1

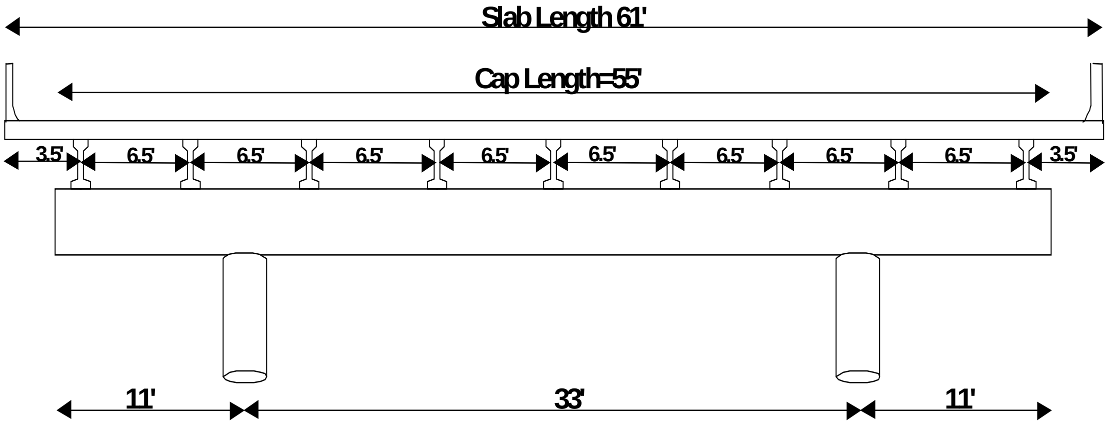


Figure 2.3 Dimensions Used in Inverted Tee Bent Analysis, IC1

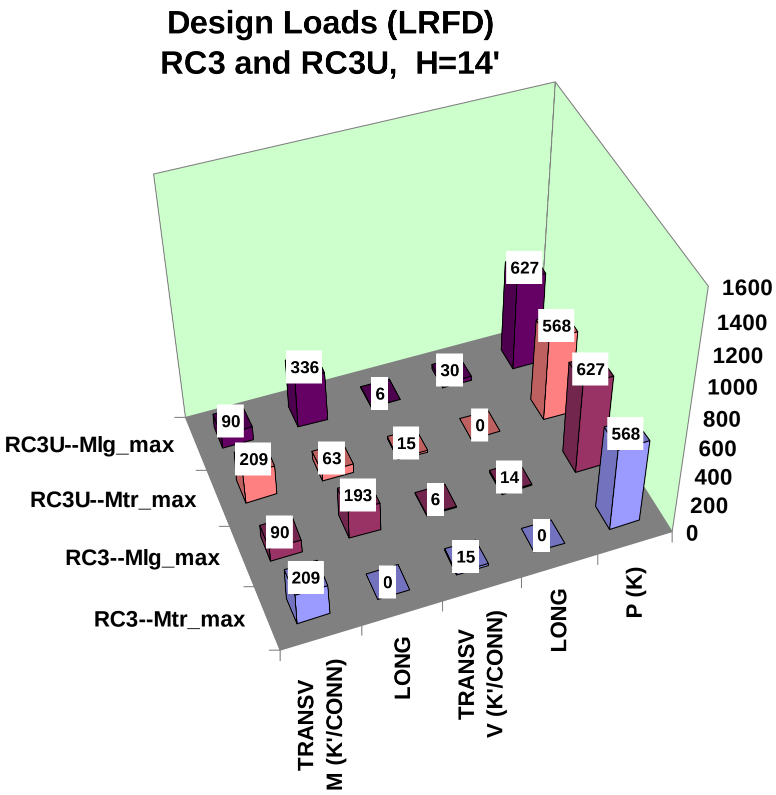


Figure 2.4a Connection Actions for Rectangular Bent Cap

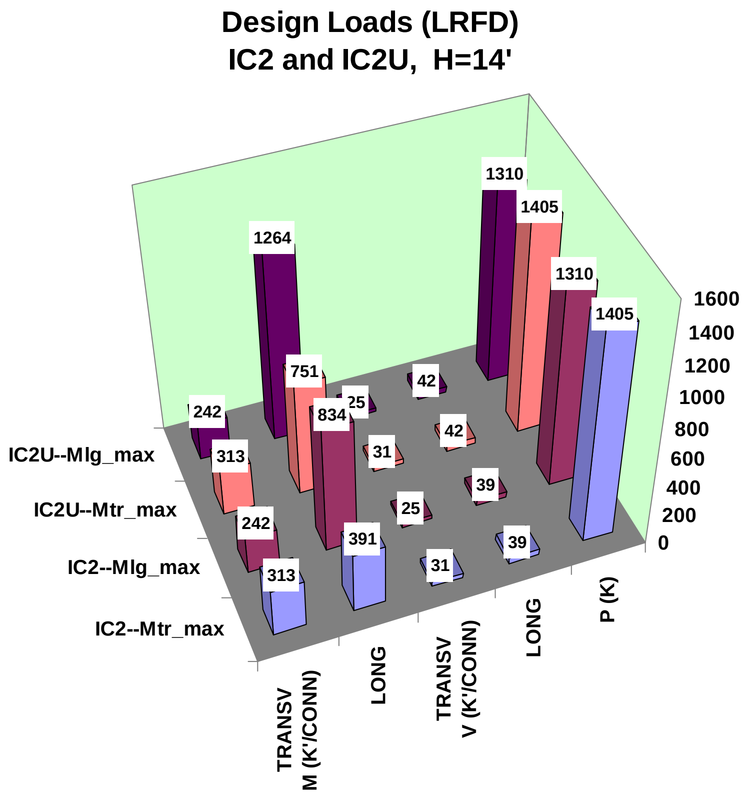


Figure 2.4b Connection Actions for Inverted Tee Bent Cap

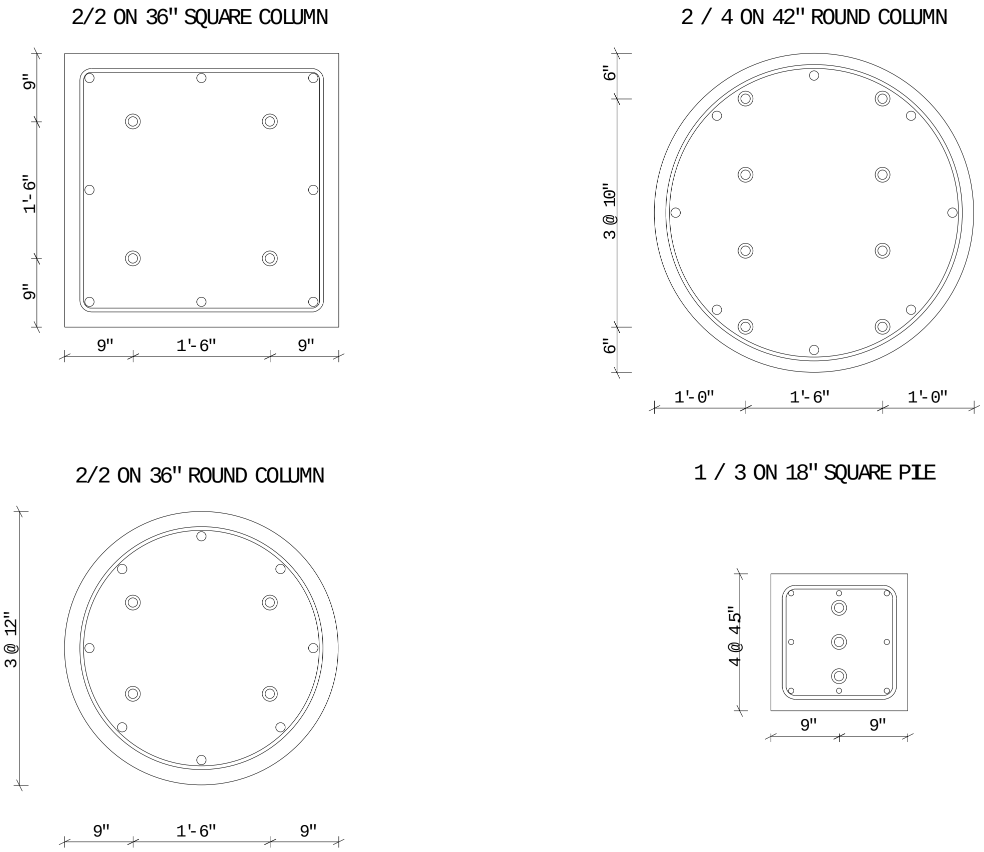
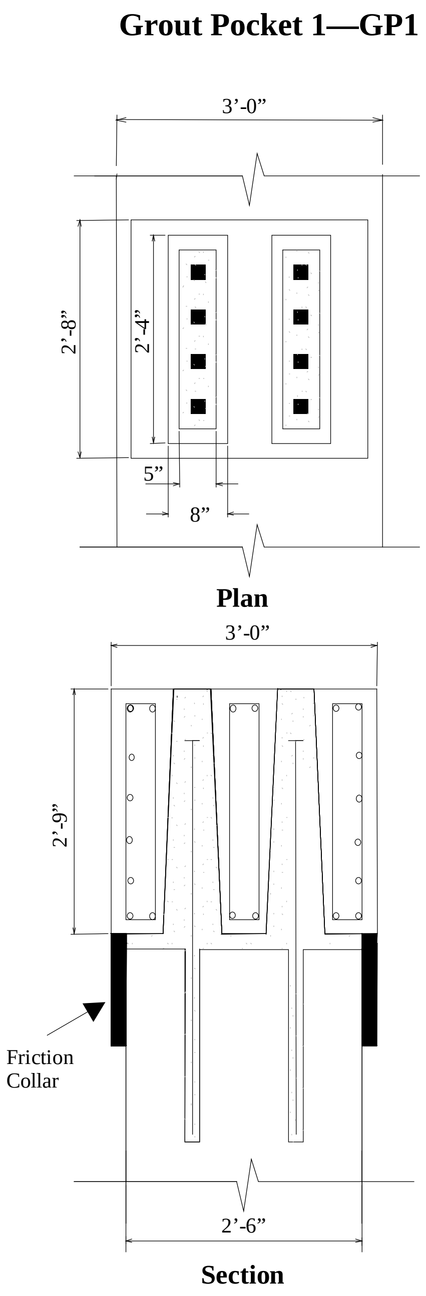


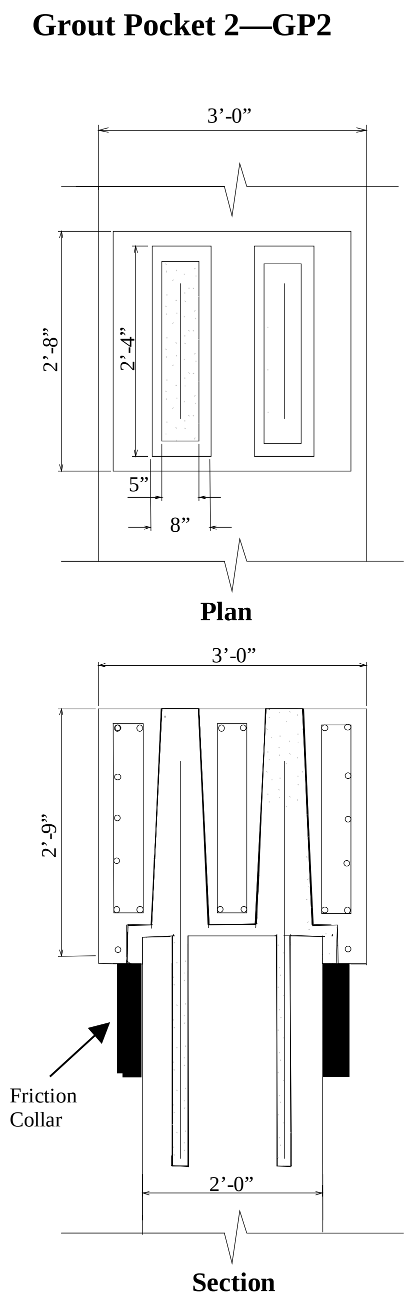
Figure 2.5 Typical Connector Arrangements



Construction Sequence

1. Drive piles with template and cut to desired elevation
2. Epoxy grout headed bars into embedded sleeves
3. Set friction collars
4. Lower cap onto collars and verify cap top elevation
5. Form around bedding layer
6. Grout bedding and pockets through cap top opening

Figure 2.6 Grout Pocket 1, GP1

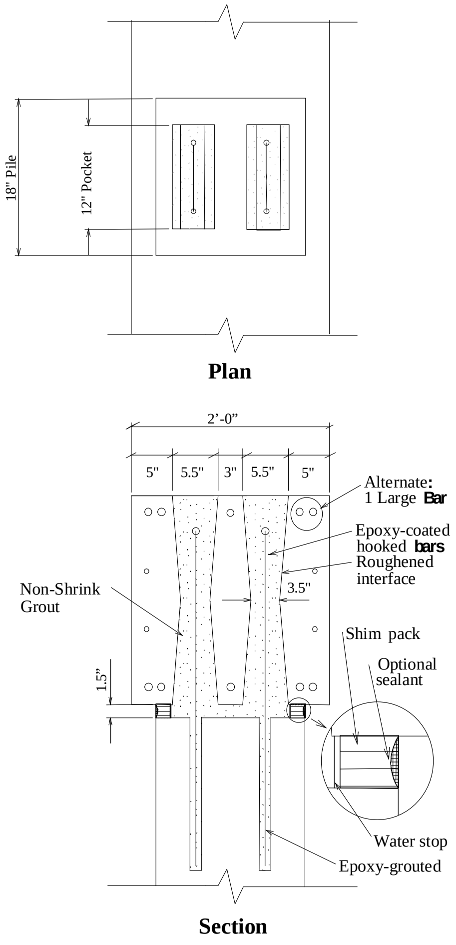


Construction Sequence

1. Drive piles with template and cut to desired elevation
2. Epoxy grout U-bars into embedded sleeves
3. Set friction collars
4. Lower cap onto collars and verify cap top elevation
5. Form around bedding layer
6. Grout bedding and pockets through cap top opening

Figure 2.7 Grout Pocket 2, GP2

Grout Pocket 1A-GP1A

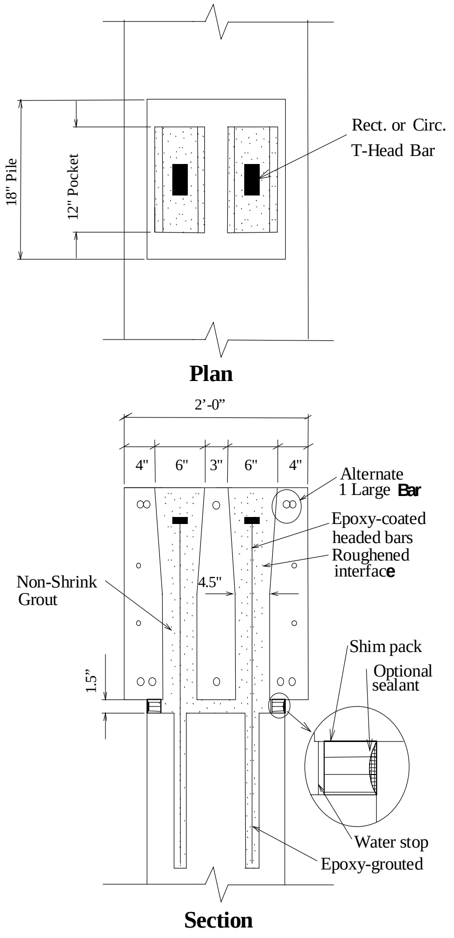


Construction Sequence

1. Drive piles with template and cut to desired elevation
2. Epoxy grout U-bars into embedded sleeves
3. Set shims
4. Lower cap onto shims and verify cap top elevation
5. Form around bedding layer; include vent tubes
6. Grout bedding and pockets through cap top opening

Figure 2.8 Grout Pocket 1A, GP1A

Grout Pocket 1B-GP1B

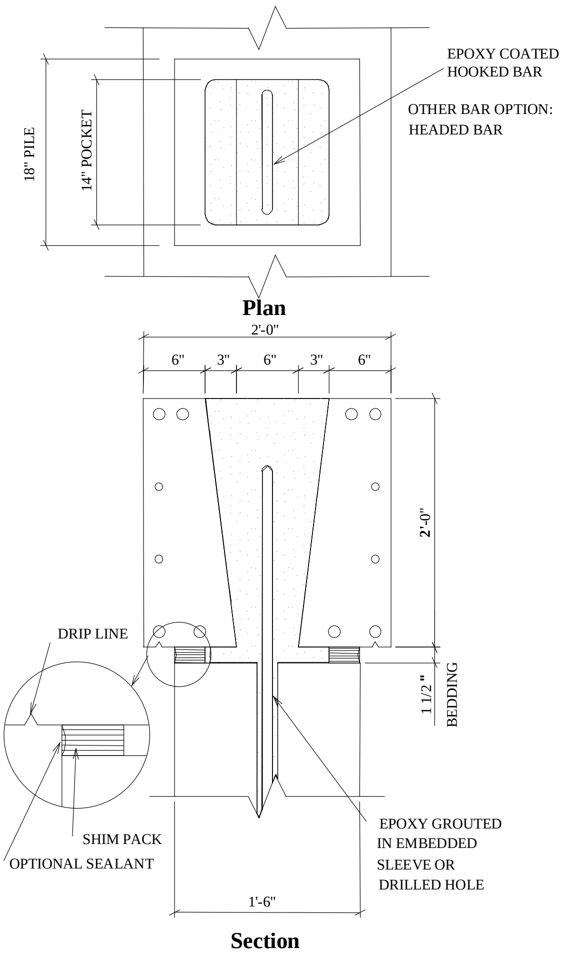


Construction Sequence

1. Drive piles with template and cut to desired elevation
2. Epoxy grout headed bars into embedded sleeves
3. Set shims
4. Lower cap onto shims and verify cap top elevation
5. Form around bedding layer; include vent tubes
6. Grout bedding and pockets through cap top opening

Figure 2.9 Grout Pocket 1B, GP1B

Grout Pocket 1C—GP1C

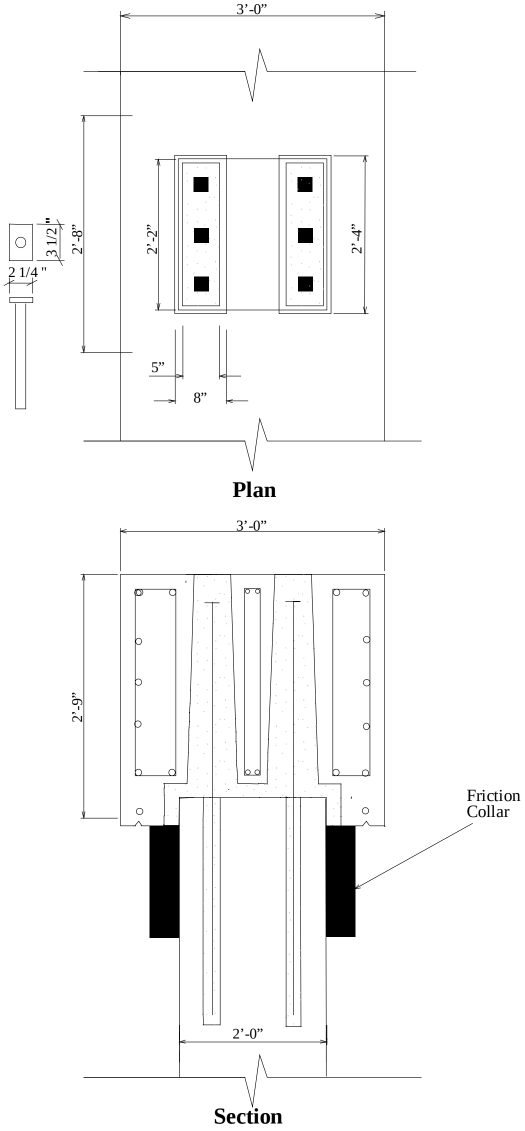


Construction Sequence

1. Drive piles with template and cut to desired elevation
2. Epoxy grout anchor bars into embedded sleeves
3. Set shims
4. Lower cap onto shims and verify cap top elevation
5. Form around bedding layer
6. Grout bedding and pockets through cap top opening

Figure 2.10 Grout Pocket 1C, GP1C

Grout Pocket 2A—GP2A

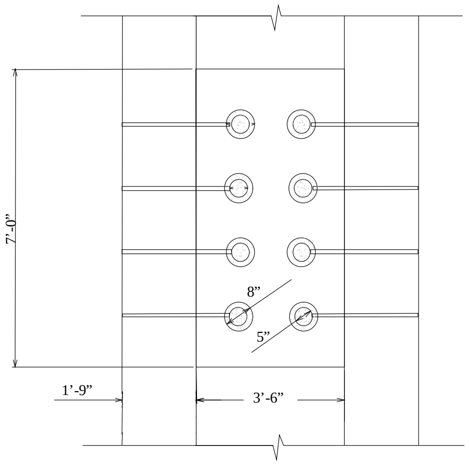


Construction Sequence

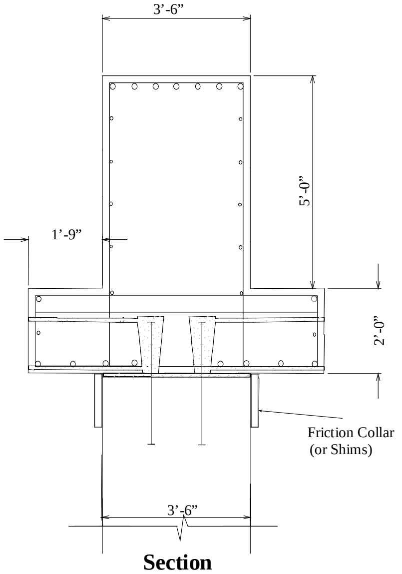
1. Drive piles with template and cut to desired elevation
2. Epoxy grout headed bars into embedded sleeves
3. Set friction collars
4. Lower cap onto collars and verify cap top elevation
5. Form around bedding layer
6. Grout bedding and pockets through cap top opening

Figure 2.11 Grout Pocket 2A, GP2A

Grout Pocket 3—GP3



Plan



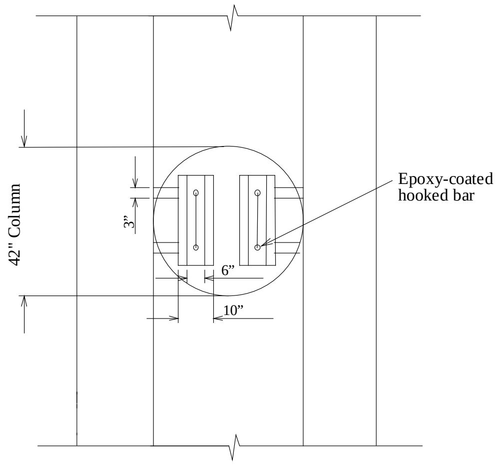
Section

Construction Sequence

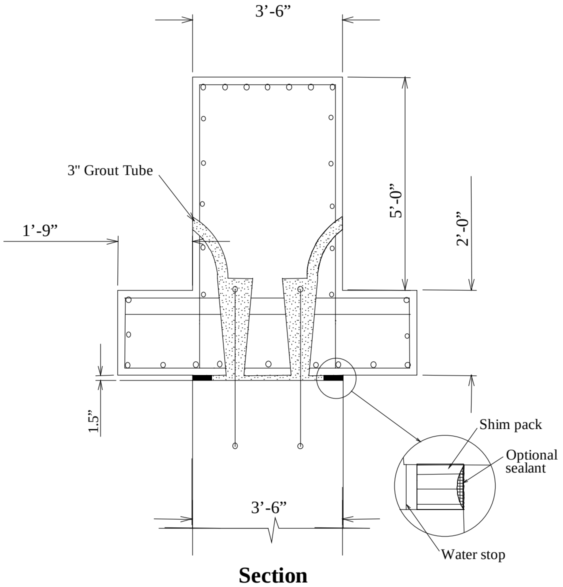
1. Cast columns in place with embedded dowels
2. Set friction collars (or shims)
3. Lower cap onto collars and verify cap elevation
4. Form around bedding layer
5. Grout pockets and bedding through horizontal ducts

Figure 2.12 Grout Pocket 3, GP3

Grout Pocket 3A—GP3A



Plan



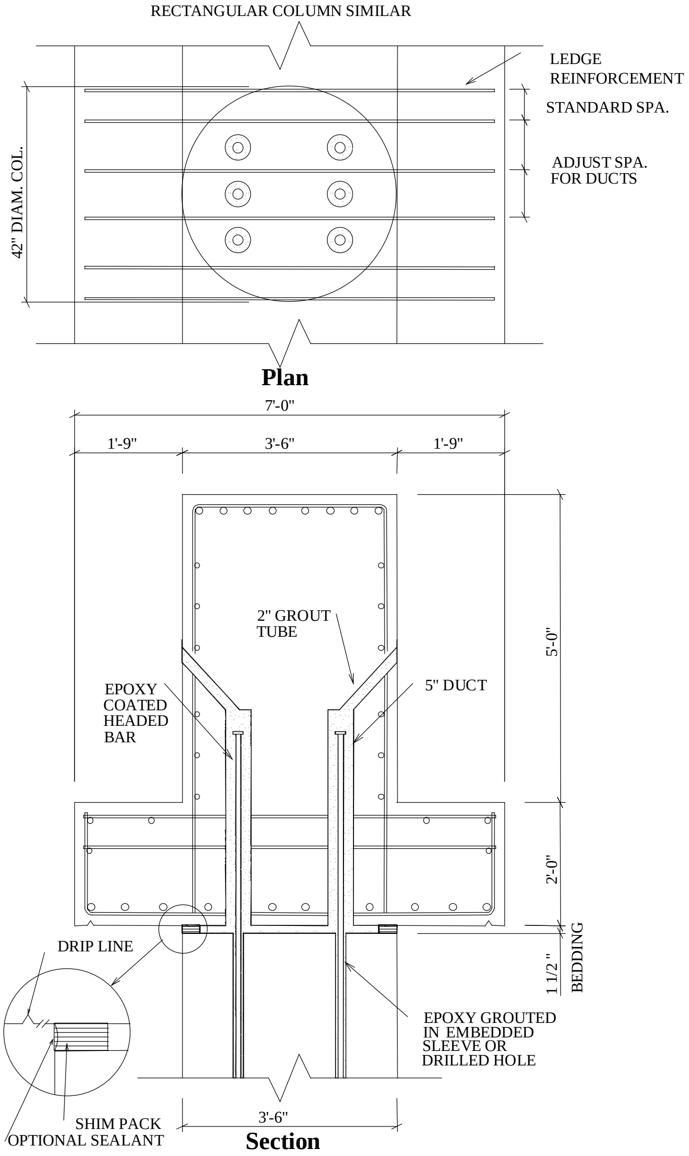
Section

Construction Sequence

1. Cast columns in place with embedded dowels
2. Set shims
3. Lower cap onto shims and verify cap elevation
4. Form around bedding layer
5. Grout bedding and pockets through grout tubes

Figure 2.13 Grout Pocket 3A, GP3A

Grout Pocket 3B—GP3B

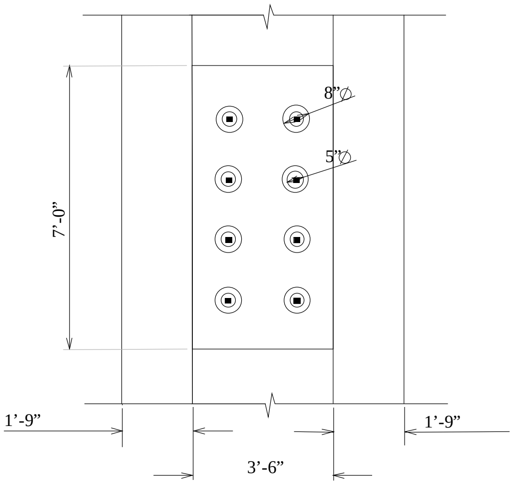


Construction Sequence

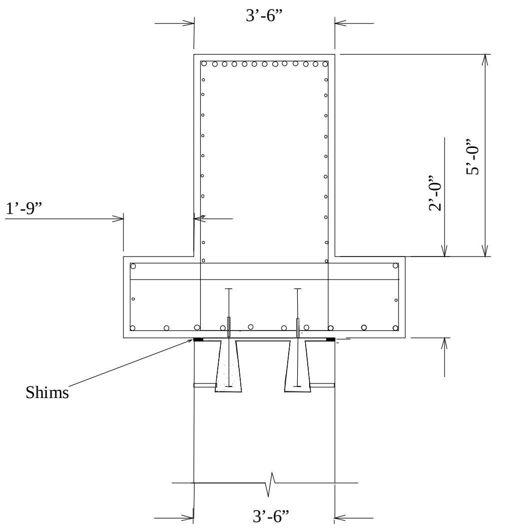
1. Cast columns in place
2. Set shims
3. Lower cap onto shims and verify cap elevation
4. Form around bedding layer
5. Grout bedding and pockets through sloped ducts

Figure 2.14 Grout Pocket 3B, GP3B

Grout Pocket 4—GP4



Plan



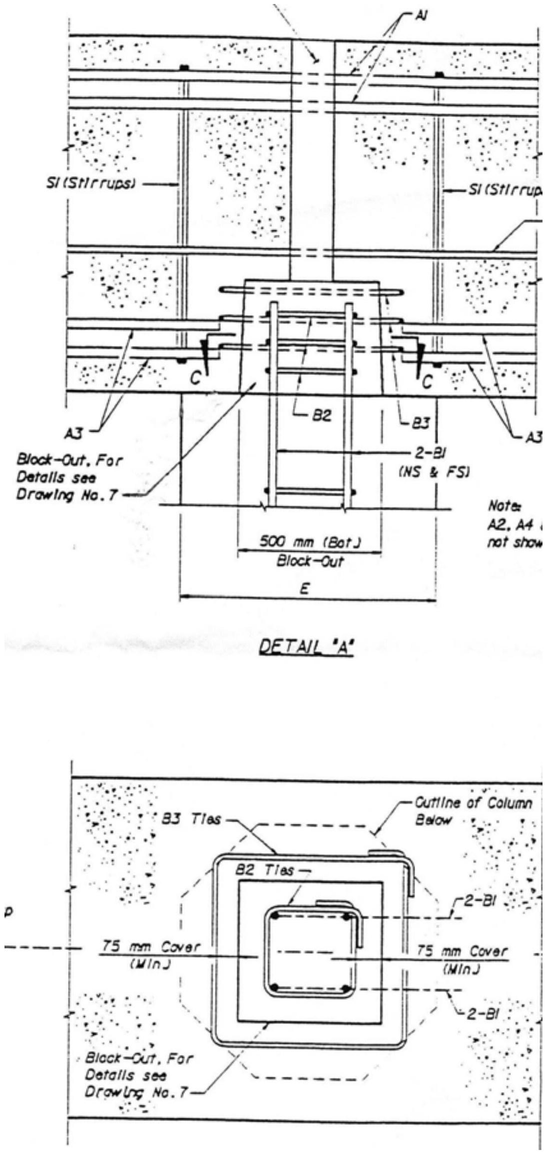
Section

Construction Sequence

1. Cast columns in place with grout pocket voids
2. Set shims
3. Attach headed reinforcement to underside of cap
4. Lower cap onto shims and verify cap elevation
5. Form around bedding layer
6. Grout bedding and pockets through sloped ducts

Figure 2.15 Grout Pocket 4, GP4

Grout Pocket 5—GP5

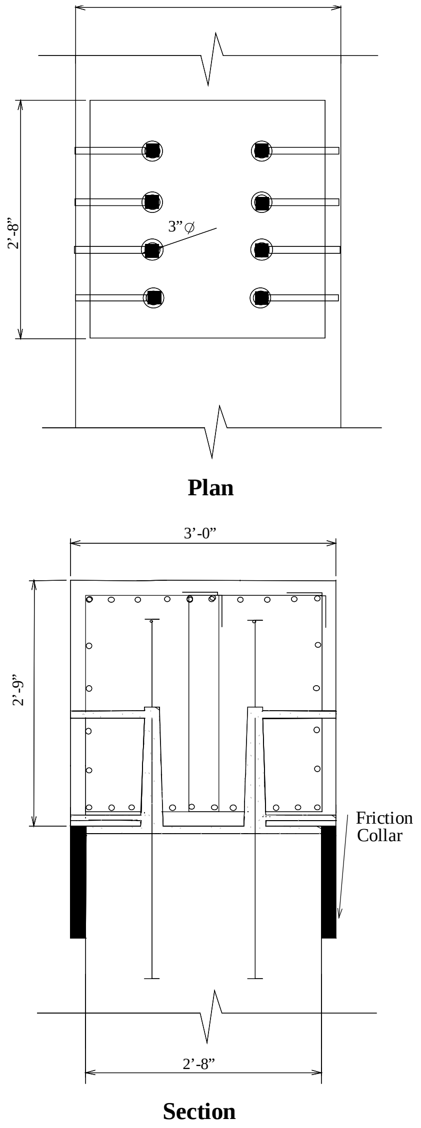


Construction Sequence

1. Cast columns in place with extended anchor bars
2. Set shims
3. Lower cap onto shims and verify cap top elevation
4. Form around bedding layer
5. Place concrete in bedding and pockets through cap top opening

Figure 2.16 Florida DOT Grout Pocket

Grouted Sleeve Coupler 1—GS1

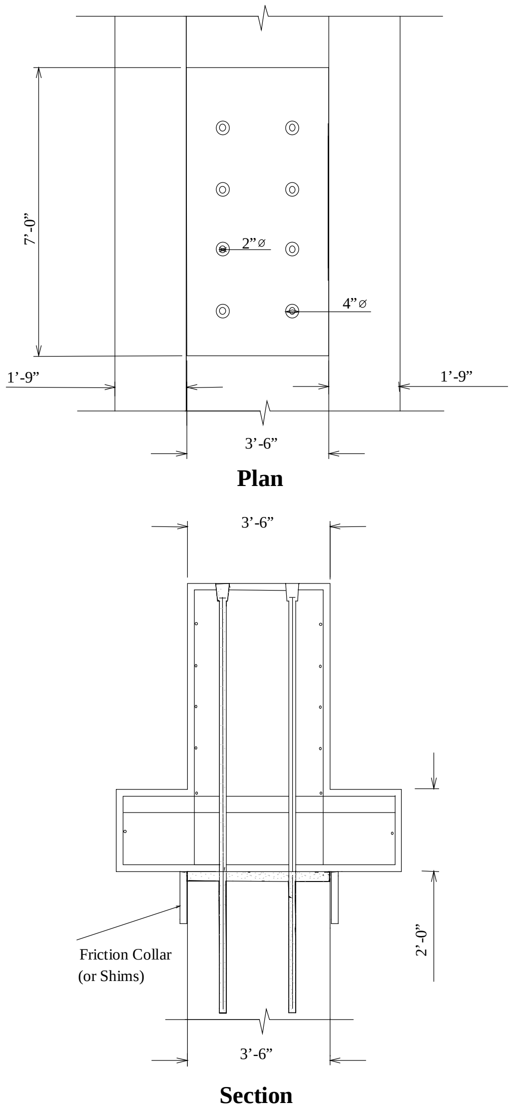


Construction Sequence

1. Cast columns in place with embedded dowels
2. Set shims
3. Lower cap onto shims and verify cap elevation
4. Form around bedding layer
5. Grout bedding
6. Pressure grout grouted sleeve couplers

Figure 2.17 Grouted Sleeve Coupler, GS1

Bolted Connection 1—BC1

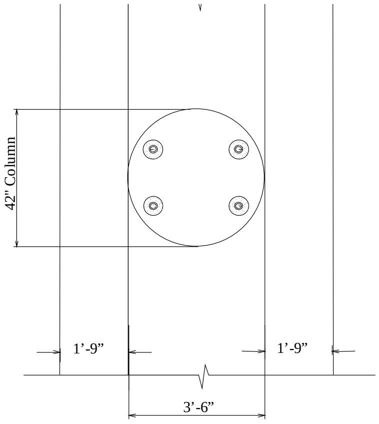


Construction Sequence

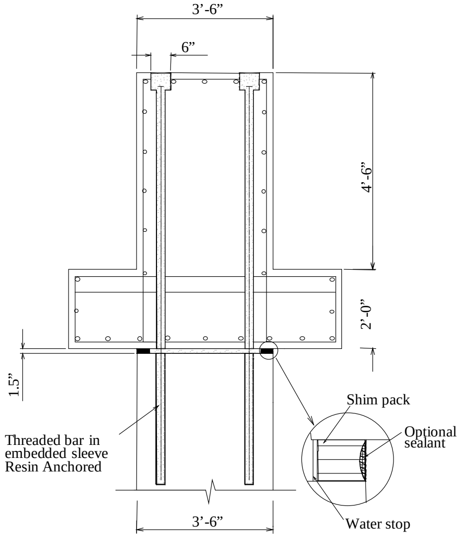
1. Cast columns in place with embedded sleeves
2. Install threadbars with resin anchor
3. Set shims
4. Thread cap over threadbars onto shims; verify elev.
5. Form around bedding layer
6. Grout bedding through bedding duct
7. Grout sheathing
8. Tighten bolts
9. Grout cap top anchorage recesses

Figure 2.18 Bolted Connection 1, BC1

Bolted Connection 1A—BC1A



Plan



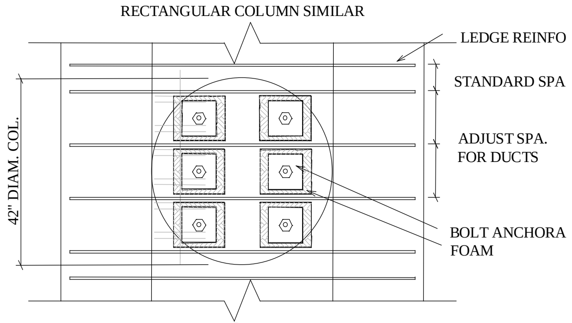
Section

Construction Sequence

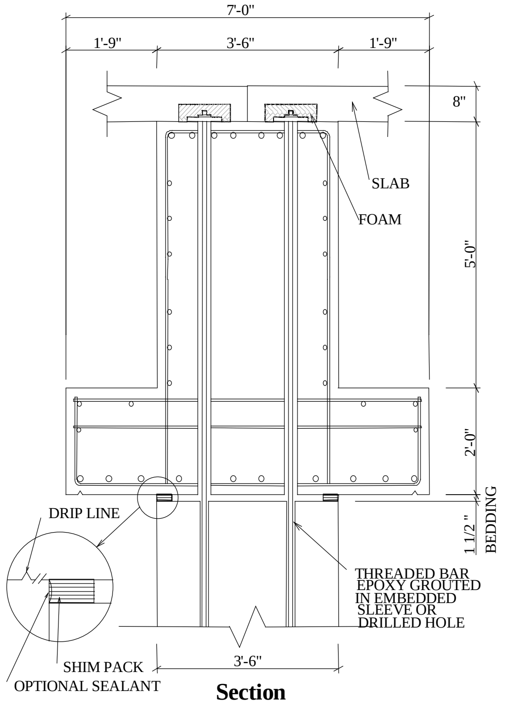
1. Cast columns in place with embedded sleeves
2. Install threadbars with resin anchor
3. Set shims
4. Thread cap over threadbars onto shims; verify elev.
3A. Lower cap onto shims and verify cap elevation
4A. Thread bolts through sheathing into threaded sleeves in column tops
5. Form around bedding layer
6. Grout bedding and sheathing through sheathing duct
7. Tighten bolts
8. Grout cap top anchorage recesses

Figure 2.19 Bolted Connection 1A, BC1A

Bolted Connection 1B—BC1B



Plan



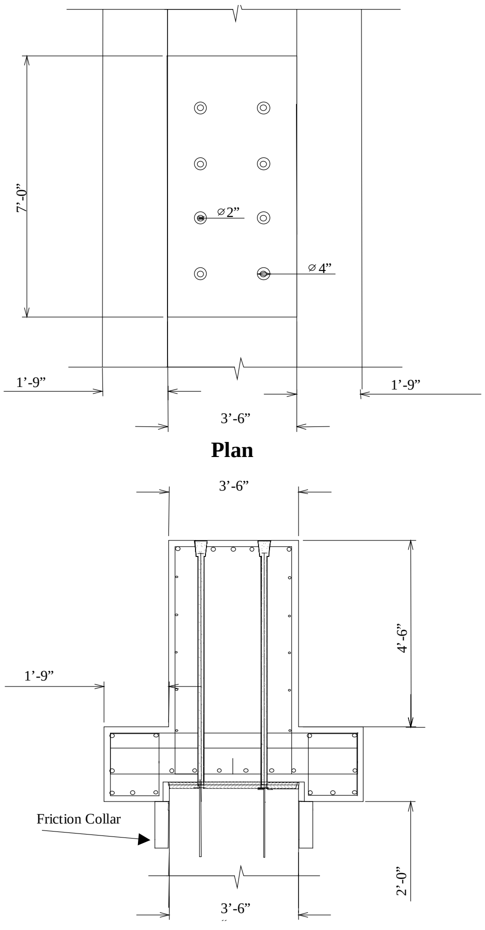
Section

Construction Sequence

1. Cast columns in place with embedded sleeves
2. Install threadbars with resin anchor
3. Set shims
4. Thread cap over threadbars onto shims; verify elev.
3A. Lower cap onto shims and verify cap elevation
4A. Thread bolts through sheathing into threaded sleeves in column tops
5. Form around bedding layer
6. Grout bedding and sheathing through sheathing duct
7. Tighten bolts

Figure 2.20 Bolted Connection 1B, BC1B

Bolted Connection 2—BC2

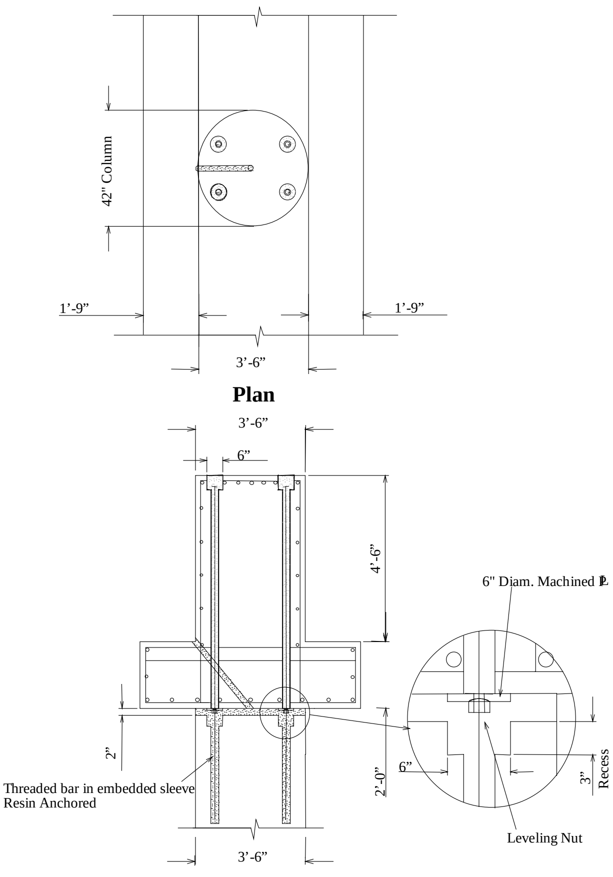


Construction Sequence

1. Cast columns in place with embedded sleeves
2. Install threadbars with resin anchor
3. Place neoprene bearing pad
4. Thread cap over threadbars and verify cap elevation
5. Tighten bolts

Figure 2.21 Bolted Connection 2, BC2

Bolted Connection 2A—BC2A



Construction Sequence

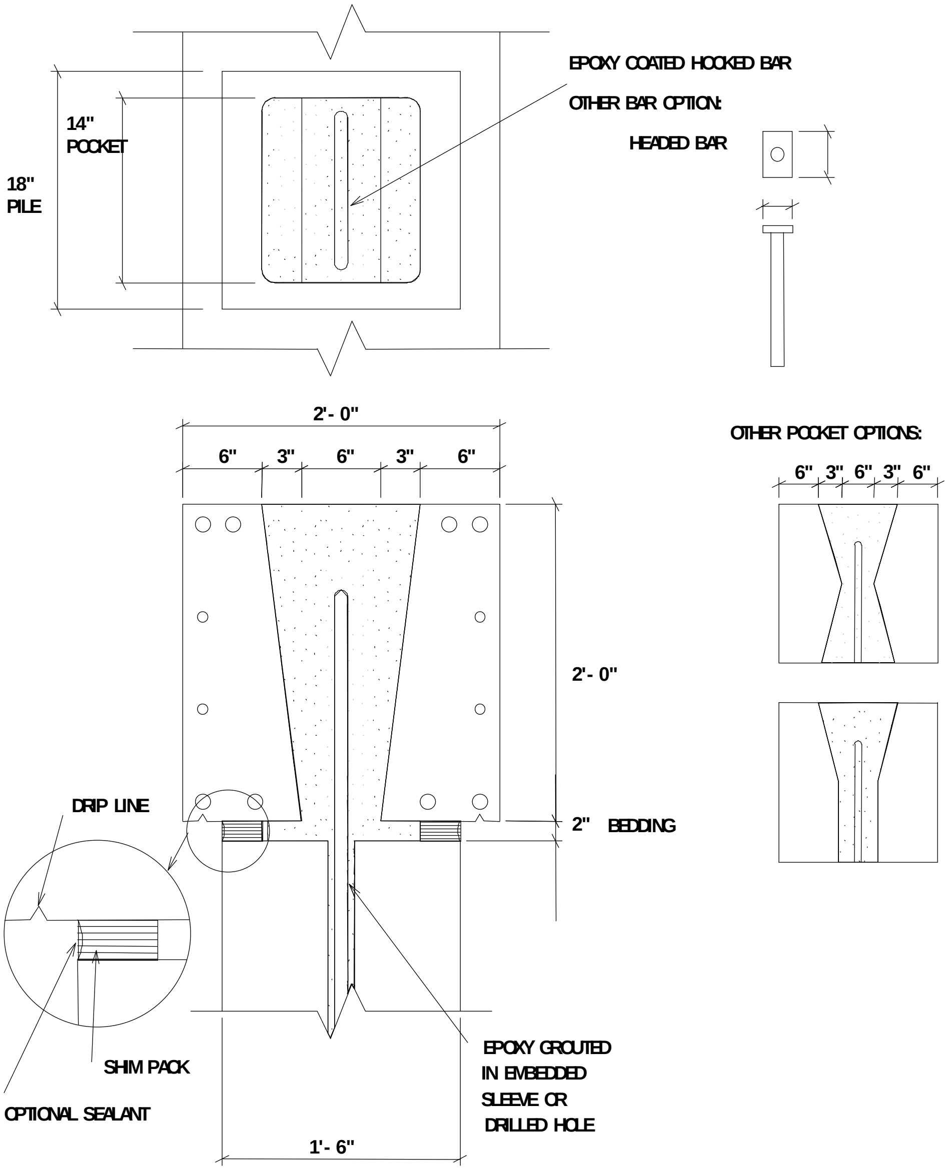
1. Cast columns in place with embedded sleeves
2. Install threadbars with resin anchor
3. Thread and adjust leveling nuts and plates to correct elevation; verify elevation
4. Thread cap over threadbars onto plates
5. Form around bedding layer
6. Grout bedding through bedding duct

6A. Grout bedding and sheathing through sheathing duct

7. Grout sheathing through sheathing duct
8. Tighten bolts

Figure 2.22 Bolted Connection 2A, BC2A

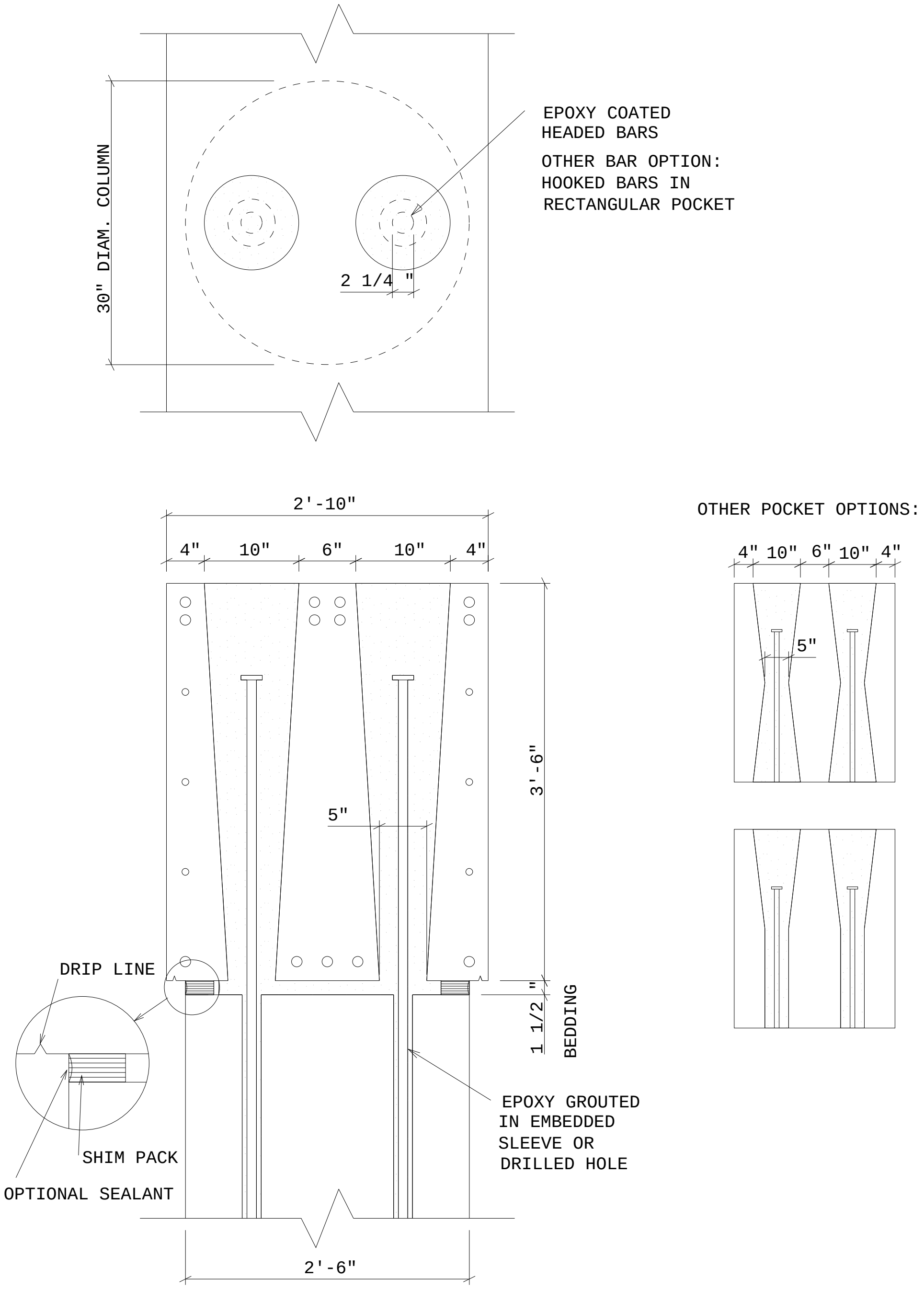
Rectangular Cap Grout Pocket on Piles



24" SQ. CAP ON 18" SQ. PILES SHOWN

Figure 2.23 Rectangular Cap Grout Pocket on Piles

Rectangular Cap Grout Pocket on CIP Column



34" x 42" CAP ON 30" DIAM. CIP COL. SHOWN

Figure 2.24 Rectangular Cap Grout Pocket on CIP Column

Inverted Tee Grout Pocket on Cast-in-Place Column

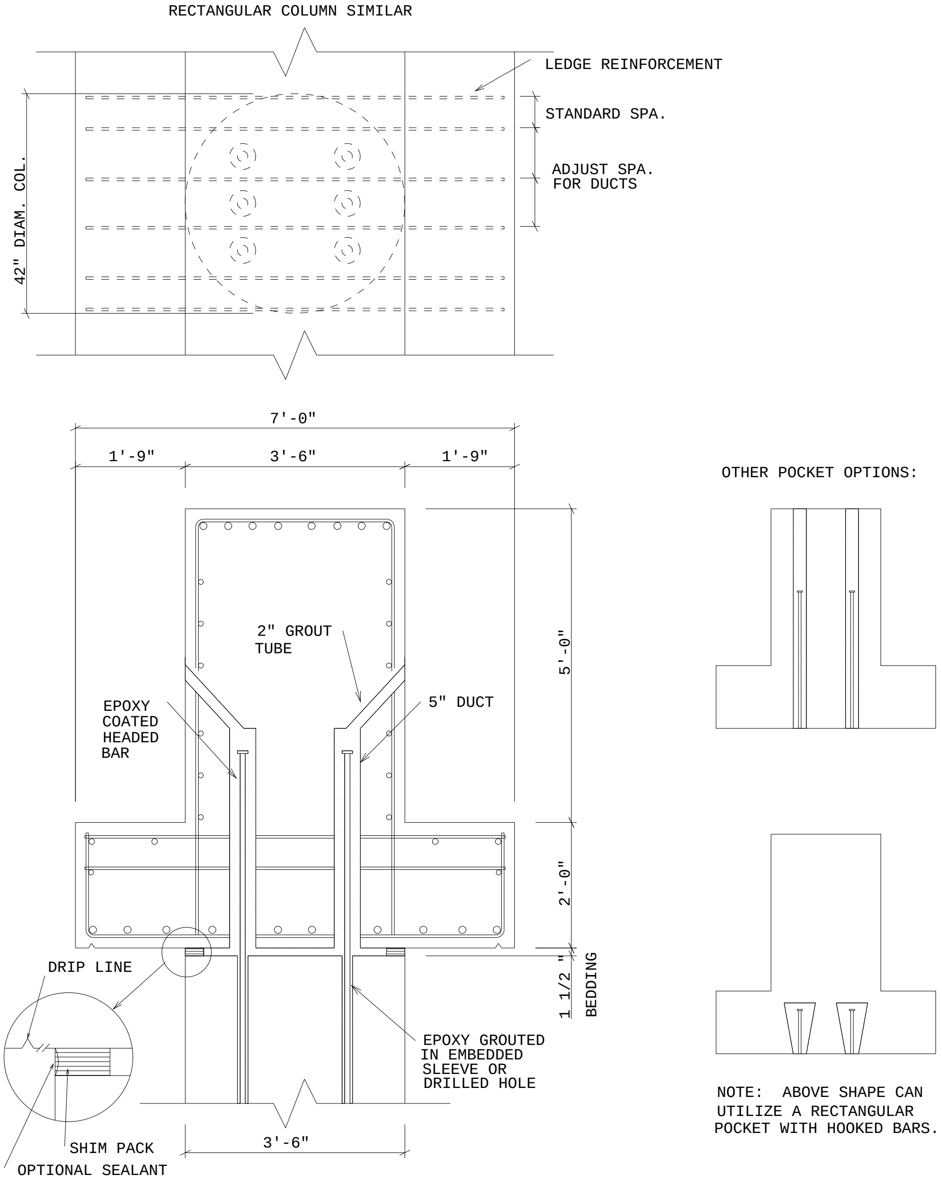


Figure 2.25 Inverted Tee Grout Pocket on Cast-in-Place Column

Inverted Tee Bolted Connection on Cast-in-Place Column

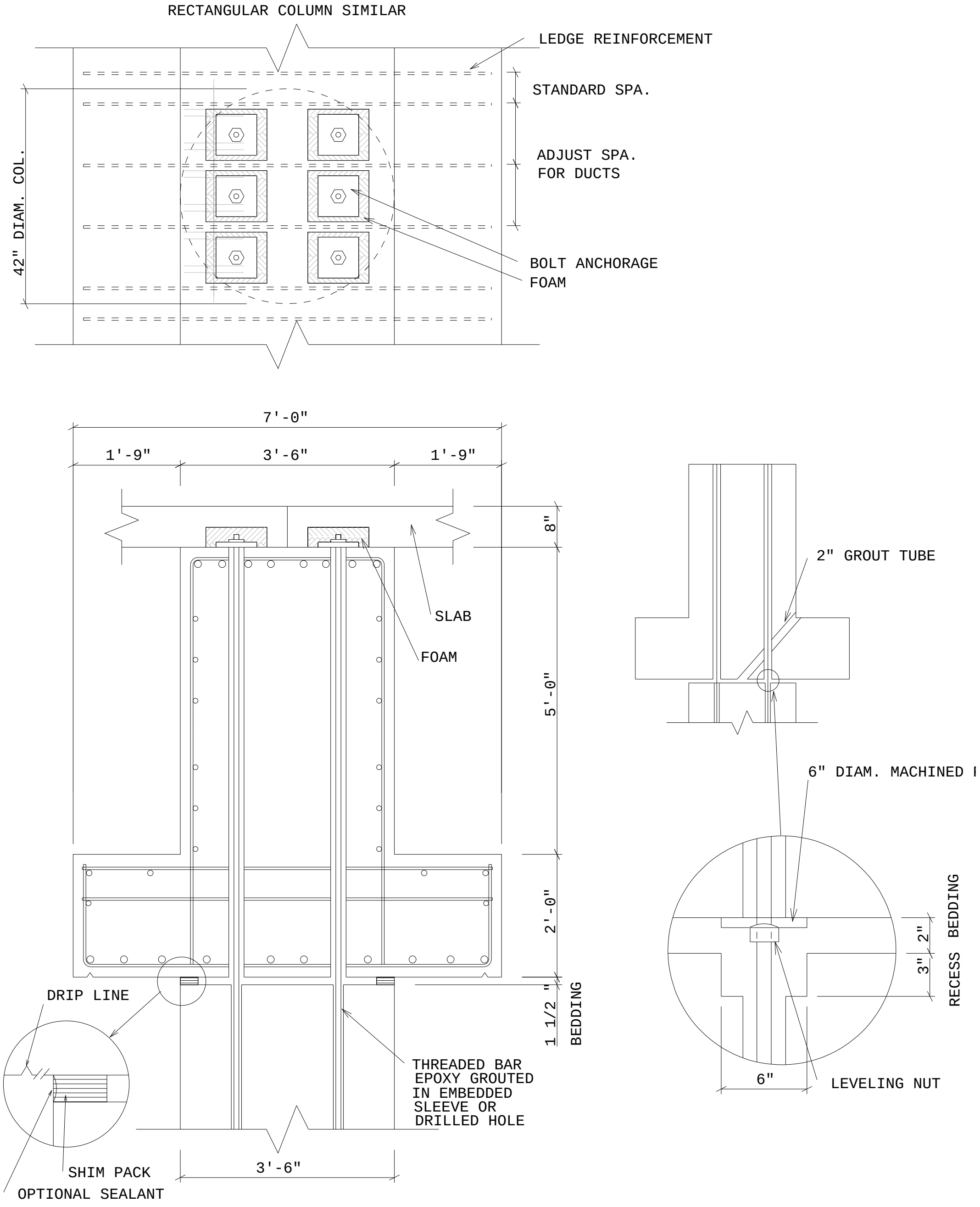


Figure 2.26 Inverted Tee Grout Pocket on Cast-in-Place column

Chapter 3: Test Program

3.1. Introduction

This chapter outlines the scope of a three-phase test program to investigate the candidate connection details discussed in Chapter 2. The goals of the test program are to develop several reliable connection details and to establish a design methodology. To accomplish this, the following objectives must be achieved: assess the influence of design parameters on the behavior and adequacy of candidate details, and verify constructability, force transfer, and, to some extent, durability of candidate connection details. These objectives will be fulfilled through three phases of testing: 1) half-scale and large-scale grout pocket pullout tests; 2) half-scale and large-scale grout pocket connection tests; and 3) large-scale grout pocket and bolted-connection tests. Phase 1 pullout tests will evaluate the influence of design parameters on the behavior and strength of connectors as they are loaded to failure. Phase 2 and 3 testing will apply realistic load combinations to bent cap connections and thus focus on verification of developed connection details. Discoveries made during testing may require modifications to the following proposed test program. The following sections describe each proposed test phase in further detail.

3.2. Phase 1—Grout Pocket Pullout Tests

Phase 1 testing will assess the influence of various design parameters on the behavior of grout pocket anchors through half-scale and large-scale pullout tests. Half-scale tests will examine response of inclined taper grout pockets for double connector lines, as shown in Figure 2.24. Full-scale tests will be conducted for the smaller precast caps that use single connector lines (Figure 2.23). Vertical duct grout pockets will be investigated through large-scale pullout tests (Figure 2.25). Possible failure modes for these grout pockets include cone pullout, side blowout, and bond/splitting failures, in addition to bar yield/fracture and crushing of concrete. The influence of the following variables on strength and failure modes will be explored: angle of taper inclination, bar type (headed versus straight), bar coating (epoxy-coated versus uncoated), bar size, bar embedment depth, concrete/grout mix, and shims. Depending on results, additional tests may be performed to determine the influence of pocket interface roughness and confining reinforcement. Conclusions regarding the behavior and adequacy of the double taper and taper with slot alternatives (Figures 2.23 and 2.24) may be formed based on the results of testing and limited finite element analyses.

Figure 3.1 shows the test matrix for the three types of test specimens during Phase 1. The majority of test specimens will use epoxy-coated bars and a small-aggregate concrete designed for enhanced-durability. Special concrete provisions include a non-shrink mix with a compressive strength between 5000 and 6000psi, a blended cement that includes fly ash for workability and reduced permeability, and pea-size (3/8”) aggregate. Calcium nitrite or other corrosion inhibitors will not be used. In contrast to the inclined taper pockets, vertical duct specimens will use a grout with fine aggregate but no pea gravel to ensure adequate grouting of the duct (and bedding layer, for later phases of testing).

The inclined taper specimen with double connector lines represents a reasonable connector arrangement for rectangular caps on cast-in-place columns. To minimize pocket interference in the cross section, inclined taper specimens with double lines incorporate a nearly vertical (one degree) taper through the cross section, with a larger, several degree taper in the transverse direction. Eight tests will be performed, with two connection tests for each specimen. Half-scale specimens will use #6 bars as connectors, approximately representing a #11 bar at half-scale. As shown in Figure 3.2, cross section dimensions will be 24” by 24”, with pockets 2.5” wide at the base, providing a horizontal tolerance of approximately +/-3/4”. Figure 3.2 also shows two connectors placed in each pocket, to account for the influence of adjacent connectors under load. Depending on specimen response, additional options may be investigated, including intentional roughening of the grout pocket-to-cap interface and the use of confinement reinforcement.

Four of the double line specimens will use epoxy-coated bars, and four will use regular, uncoated bars. Comparisons between behavior of epoxy-coated and uncoated bars will be useful, as some designs may not require epoxy-coated bars. In addition, response for a limited number of uncoated bars provides a means to compare results with the larger body of available test data on anchorage of uncoated bars. As shown in the test matrix, upset headed bars and straight bars will be tested. The newly developed HRC upset headed bar provides anchorage by bearing of a head that has a diameter ~1.5 times that of the shaft and by bond along a shaft with standard bar deformations. Because the head is considerably smaller than the HRC standard *T-Headed* bars, tighter tolerances associated with precast bent caps may be more easily accommodated. If necessary, plates may be attached to the upset head to enhance anchorage for select tests. Testing of straight bars provides a baseline to compare the effectiveness of anchorage for headed bars versus straight bars and will also help determine the capacity of standard, straight bars in grout pockets. The influence of embedment depth will also be studied for the epoxy-coated connectors. Testing with two depths will help determine a range of reasonable embedment depths to develop the strength of bars and may provide data for different failure modes. Tests on specimens with uncoated bars will examine response for two types of concrete—the "standard" prepackaged mix described earlier and a second mix designed for added durability protection—and for a single embedment depth.

A second series of inclined taper tests, which use a single connector line with a larger angle of inclination, are also shown in Figure 3.1. This connection alternative would be feasible for cases such as precast caps on trestle piles where a small number of connectors are required. Reasonably large horizontal tolerances of approximately +/-2" may be accommodated, while also incorporating a taper angle as large as approximately five degrees. These tests will assess the influence of the larger "wedge" along a single line of connectors. Four tests will be performed using two specimens. Full-scale specimens will be used, with #8 bars as connectors. Figure 3.3 shows the 24" by 24" square cross section, with a pocket 5" wide at the base. Figure 3.3 also shows two connectors placed along the single connector line. Epoxy-coated, upset-headed bars and epoxy-coated, straight bars will be tested at two different depths.

The final series of pullout tests are four vertical duct tests. Specimens will use full-scale, standard steel corrugated post-tensioning ducts and connectors, because of the uncertainty in scaling effects with respect to confinement for corrugated ducts. Ducts will have an inner diameter of 4", providing a horizontal tolerance of approximately +/-1". Epoxy-coated, upset-headed #11 bars and straight #11 bars will be tested at two depths for each type of connector. Figure 3.4 shows the 24" wide by 30" deep cross section. Single ducts will be tested, as the influence of an individual connector on adjacent ducts may be limited. Multiple ducts may be investigated later if deemed necessary.

A test set-up will be constructed to permit monotonic, quasi-static tension loading of test specimens to failure. As shown in Figure 3.5, a spreader beam will be used with center hole hydraulic rams to minimize confinement effects on connectors due to bearing of reactions. To represent group action behavior of connectors during double line connector tests, two rams will be used simultaneously to load two connectors. Initial testing will involve loading two connectors in the same pocket, while later testing may investigate response when loading one connector in each pocket. Both connectors will be loaded for single line connector tests. As a minimum, instrumentation will include a load cell/pressure transducer, slip wires, linear transducers, dial gauges, and strain gauges. This will permit load-deflection and stress-strain response of the connectors to be determined during testing.

Although force transfer issues are of primary interest during Phase 1 testing, constructability and durability issues will also be addressed to a limited extent. Constructability will be assessed throughout construction of specimens. In addition, the extent and influence of shrinkage-related cracking within pockets will also be monitored to evaluate potential impact to durability.

3.3. Phase 2—Grout Pocket Connection Tests

Phase 2 testing will assess the adequacy of grout pocket connection details for rectangular and inverted-tee bent caps through half-scale and full-scale connection tests. In contrast to the Phase 1 pullout tests, Phase 2 testing will involve the application of realistic bent cap loading to complete bent cap-to-column connections and therefore may serve to verify representative connection details.

The more modest test matrix for Phase 2 is shown in Figure 3.6. Testing will be limited to six tests involving epoxy-coated, upset headed and straight bars with durability-enhanced concrete or grout. It is anticipated that Phase 1 results will help determine certain test parameters for Phase 2, such as embedment depth for bars. Construction of test specimens will be considerably simplified by designing specimens to the same scale as for Phase 1. In addition to allowing reuse of formwork, this will permit better correlation of test results for Phases 1 and 2.

Figure 3.7 shows the more complex loading to be used in this phase. Vertical loads will be applied at relatively small eccentricities to connection specimens supported by stub columns. The use of multiple rams will permit the application of loads at a transverse eccentricity (rams 1 and/or 2), a longitudinal eccentricity (rams 3 and 4), or both eccentricities simultaneously. This will enable realistic axial load and bending combinations about each axis to be applied. Analyses summarized in Chapter 2 suggest that relatively small eccentricities are expected for most cases of bent cap loading, implying concrete crushing at the cap-to-column interface as a likely failure mode. Instrumentation for Phase 2 will be very similar to that of Phase 1.

In addition to enabling realistic loads to be applied to specimens, modeling of an entire cap-to-column connection provides other important advantages: 1) suitability of gravity-flow grouting may be investigated; 2) practical limits for the bedding layer depth may be examined; 3) effects of load concentration due to use of shims in the bedding layer may be assessed; 4) confinement effects related to bearing at the bedding layer may be included in testing; 5) cracking at the bedding layer due to possible grout-related expansion or applied loads may be observed; and 6) the effectiveness of sealants may be tested if necessary. Issues not resolved through this phase of testing may be further addressed in the large-scale connection testing of Phase 3.

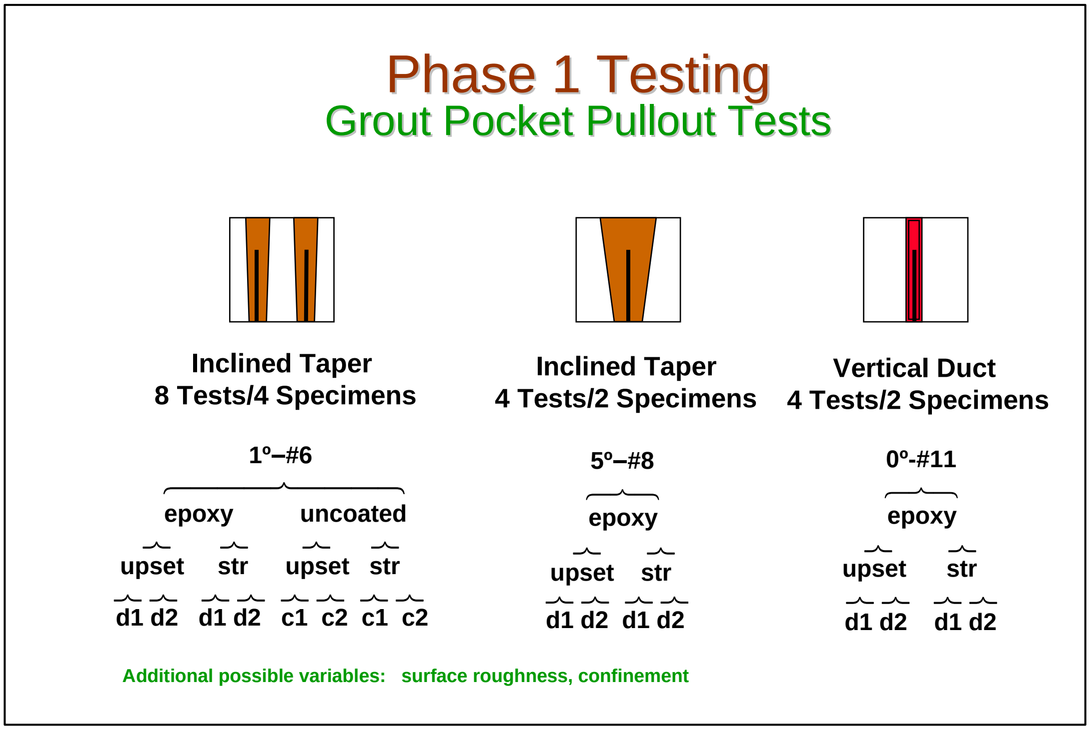
3.4. Phase 3—Large-scale Connection Tests

Phase 3 testing will assess the adequacy of select grout pocket and bolted connection details through limited full-scale bent cap-to-column connection tests. Based on results of Phases 1 and 2, large-scale grout pocket and bolted connection details will be constructed and tested with the assistance of industry representatives. The scope of this phase will be determined in coordination with the Industry Review Committee. Important constructability issues such as grout pocket forming, gravity-flow grouting, practical limits for bedding layer thickness, and overall ease of construction may be verified by construction of large-scale specimens. This may be critical if half-scale testing does not fully resolve major constructability concerns. Large-scale testing will also allow cracking at the bedding layer and grout pockets to be investigated. Issues specific to bolted connections, such as adequacy of anchorage schemes and adjustability of double nuts, may also be investigated. If possible, large-scale specimens will be loaded to ultimate levels to provide proof testing and confirm failure modes.

3.5. Test Program Schedule

Figure 1.1 showed all phases of testing scheduled for completion by the second quarter of FY 1999. After construction of Phase 1 specimens during the third quarter of FY 1998, testing is scheduled for the fourth quarter of FY 1998. Phase 2 construction and testing is planned for the first and second quarters of FY 1999. Industry-assisted Phase 3 construction and testing will be conducted concurrently with Phase 2 efforts. The last two quarters of FY 1999 will be used to reduce and analyze data and produce final connection details, a design methodology, and the final report.

Chapter 3 Figures



Phase 1 Testing

Grout Pocket Pullout Tests

Inclined Taper
8 Tests/4 Specimens

1º–#6

epoxy — **upset**: **d1 d2**; **str**: **d1 d2**

uncoated — **upset**: **c1 c2**; **str**: **c1 c2**

Inclined Taper
4 Tests/2 Specimens

5º–#8

epoxy — **upset**: **d1 d2**; **str**: **d1 d2**

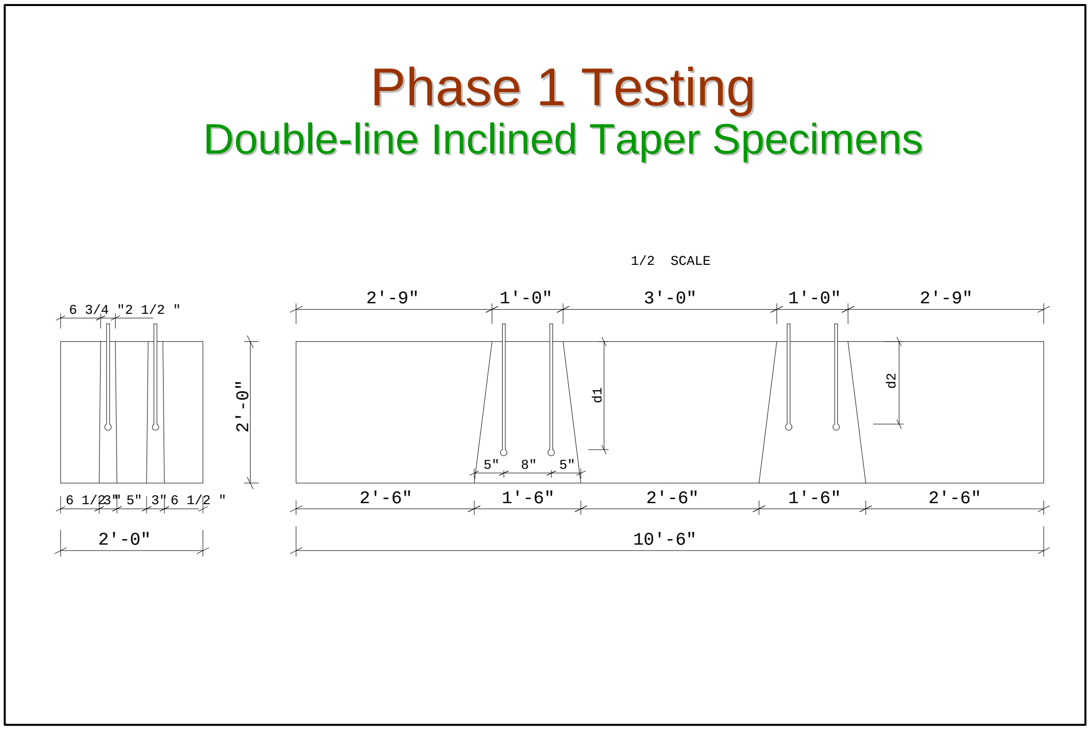
Vertical Duct
4 Tests/2 Specimens

0º-#11

epoxy — **upset**: **d1 d2**; **str**: **d1 d2**

Additional possible variables: surface roughness, confinement

Figure 3.1 Phase 1 Test Matrix



Phase 1 Testing

Double-line Inclined Taper Specimens

Figure 3.2 Phase 1 Double Line Inclined Taper Specimens

Phase 1 Testing

Single-line Inclined Taper Specimens

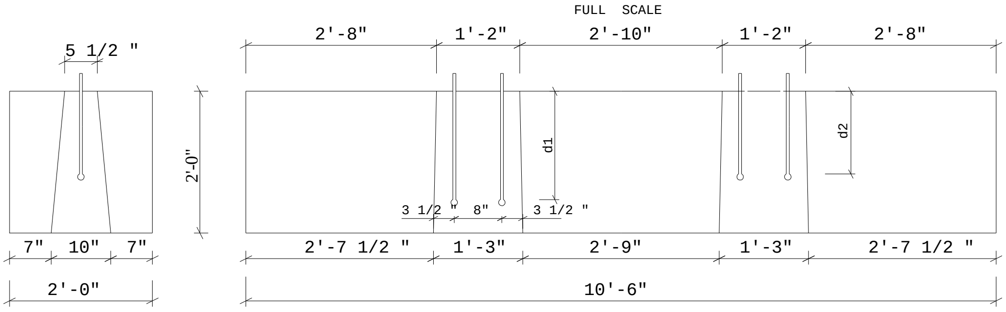


Figure 3.3 Phase 1 Single Line Inclined Taper Specimens

Phase 1 Testing

Vertical Duct Specimens

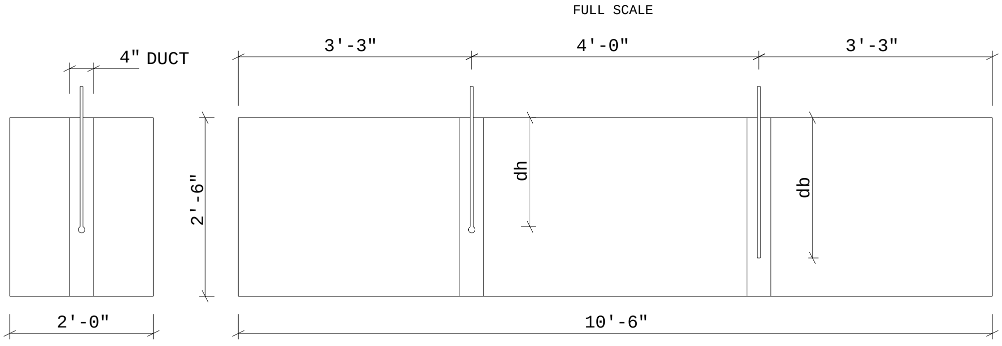
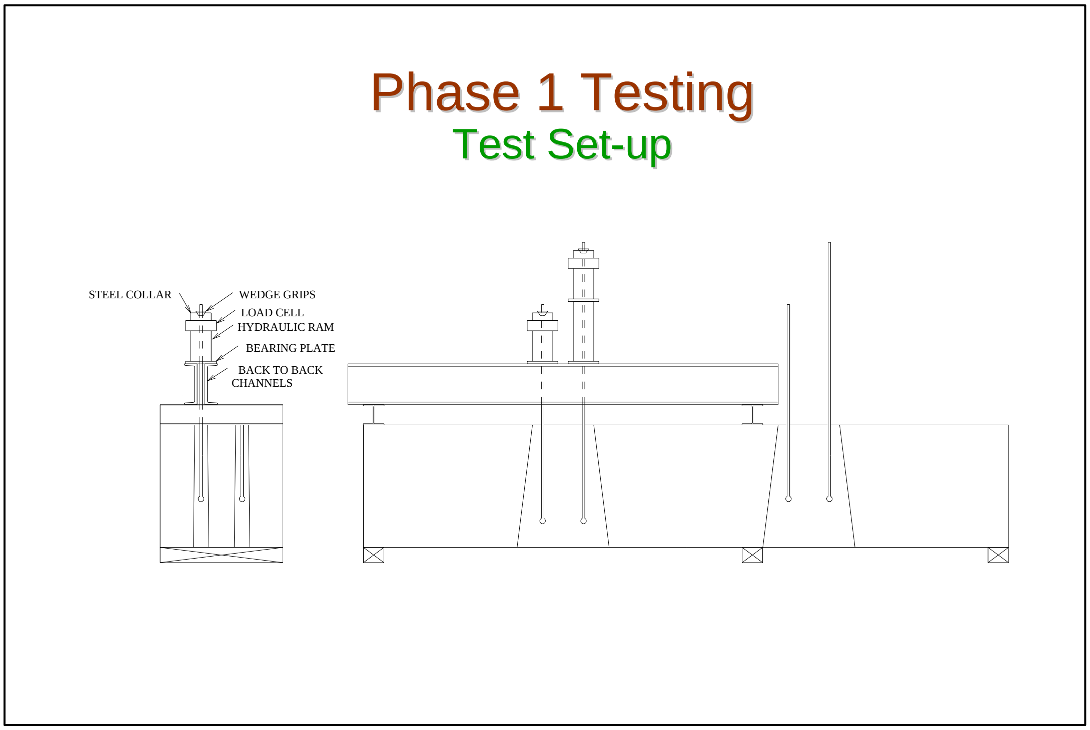


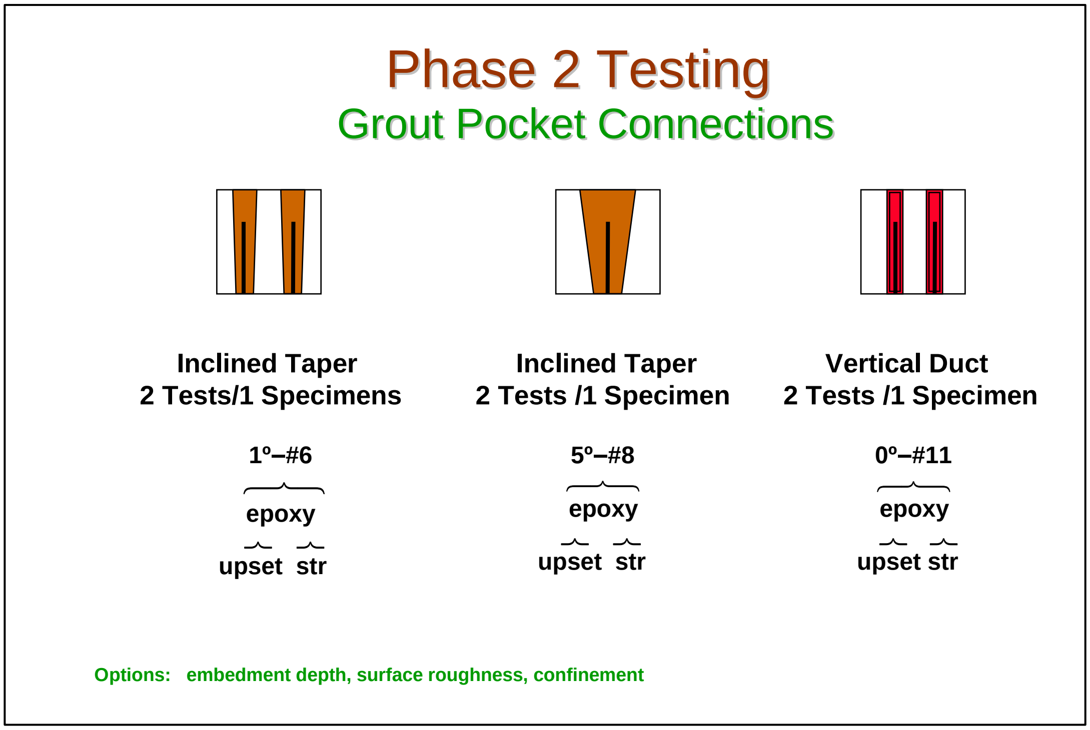
Figure 3.4 Phase 1 Vertical Duct Specimens



Phase 1 Testing

Test Set-up

Figure 3.5 Phase 1 Test Setup



Phase 2 Testing

Grout Pocket Connections

Inclined Taper
2 Tests/1 Specimens

1º–#6
epoxy
upset str

Inclined Taper
2 Tests /1 Specimen

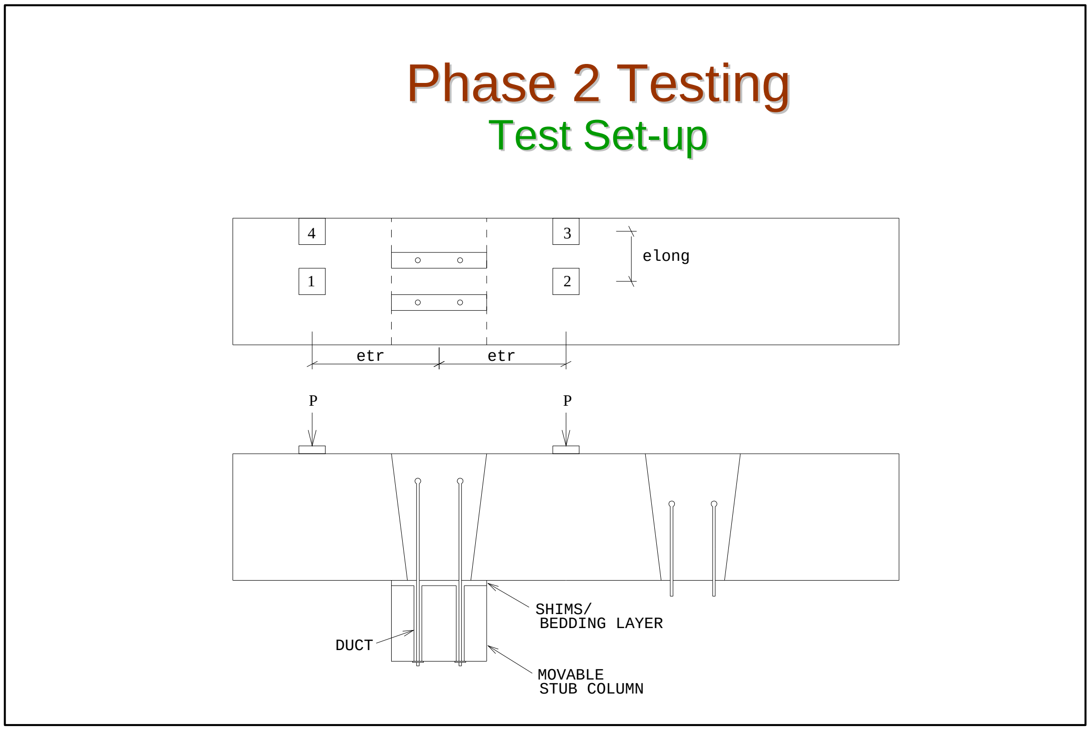
5º–#8
epoxy
upset str

Vertical Duct
2 Tests /1 Specimen

0º–#11
epoxy
upset str

Options: embedment depth, surface roughness, confinement

Figure 3.6 Phase 2 Test Matrix



Phase 2 Testing

Test Set-up

Figure 3.7 Phase 2 Test Setup

REFERENCES

1. LoBouno, Armstrong and Associates (LAA), “Development of Precast Bridge Substructures,” Report to the Florida Department of Transportation, May 1996.

2. Wolf, Lloyd M., and Norman K. Friedman, “Red Fish Bay and Morris and Cummings Cut: Innovations in Bridge Construction and Durability,” Technical Quarterly, v. 9, No. 2, October 1994, Texas Department of Transportation, pp. 1-3.

3. Red Fish Bay and Morris and Cummings Cut Bridges (shop drawings), Heldenfels Enterprises, March 1994.

4. Etcheverry, Leandro et al., “Evaluation of the IH-45 Pierce Street Overhead,” Research Report, Center for Transportation Research, The University of Texas at Austin, December 1995.

5. Pierce Street Elevated Bridge (shop drawings), Traylor Brothers, Inc., October 1996.

6. Texas State Railroad (design drawings), Texas Department of Transportation, Bridge Design Division, June 1995.

7. Van der Veen, T., “Precasting the Edison Bridge,” PCI Convention, Nashville, October 1992.

8. Texas State Department of Highways and Public Transportation (TSDHPT), *Bridge Design Guide* (1st ed.), TSDHPT, Austin, 1990.

9. American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges* (15th ed.), AASHTO, Washington, D.C., 1992.

10. American Association of State Highway and Transportation Officials (AASHTO), *AASHTO LRFD Bridge Design Specifications: Customary U.S. Units* (1st ed.), AASHTO, Washington, D.C., 1994.