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16. Abstract

Improved speed of construction and economy can be achieved through the use of precast bridge substructures. As a step in the advancement of precast bridge substructures, a precast bent cap system is developed for nonseismic regions, including a design methodology, construction guidelines, and example details for connecting a precast bent cap to cast-in-place columns or precast trestle piles. Three categories of connection details are developed: grout pockets, grouted vertical ducts, and bolted connections.

Three phases of construction and testing of the three connection types are reported. The first phase examines behavior and failure modes for grout pocket and grouted vertical duct connections through thirty-two large-scale pullout tests. Results of four full-scale bent cap-to-column connection assemblages are presented for the second phase. In addition, results of construction and testing for two full-scale bents at the construction yard of a highway contractor are provided.

Based on test results, equations for anchorage of straight or headed connectors in grout pocket or grouted vertical duct connections are established. Design recommendations are presented in code format, and construction guidelines are presented in the form of a precast connection specification. Example connection details that conform to the design requirements and the precast connection specification are shown for each connection type.

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DEVELOPMENT OF A PRECAST BENT CAP SYSTEM

by

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M. E. Kreger, S. L. Wood, and J. E. Breen

Research Report 1748-2

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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
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DISCLAIMER

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SUMMARY

Improved speed of construction and economy can be achieved through the use of precast bridge substructures. As a step in the advancement of precast bridge substructures, a precast bent cap system is developed for nonseismic regions, including a design methodology, construction guidelines, and example details for connecting a precast bent cap to cast-in-place columns or precast trestle piles. Three categories of connection details are developed: grout pockets, grouted vertical ducts, and bolted connections.

Three phases of construction and testing of the three connection types are reported. The first phase examines behavior and failure modes for grout pocket and grouted vertical duct connections through thirty-two large-scale pullout tests. Results of four full-scale bent cap-to-column connection assemblages are presented for the second phase. In addition, results of construction and testing for two full-scale bents at the construction yard of a highway contractor are provided.

Based on test results, equations for anchorage of straight or headed connectors in grout pocket or grouted vertical duct connections are established. Also, a design methodology for a precast bent cap system is developed that includes the following eight steps: 1) selection of a trial bent configuration, 2) analysis of bent cap and columns, 3) determination of connection actions, 4) selection of connection type and embedment, 5) selection of trial connector configuration, 6) analysis of connector configuration, 7) determination of connector type and embedment, and 8) determination of confining reinforcement and auxiliary reinforcement. Construction guidelines are established based on construction experiences during the three phases of research.

Lastly, design recommendations are presented in code format, and construction guidelines are presented in the form of a precast connection specification. Example connection details that conform to the design requirements and the precast connection specification are shown for each connection type.

CHAPTER 1:
INTRODUCTION

1.1 BACKGROUND

1.1.1 General

The impact and costs associated with traffic control and disrupted traffic flow have increased significantly in recent years as the Texas Department of Transportation (TxDOT) continues to build bridges in increasingly congested urban environments. Concerns about traffic delays as well as public and worker safety in construction zones have heightened as traffic volume has increased. Additionally, direct and indirect costs related to traffic control, disruption, and environmental impact have become a major concern.

To help address these concerns, TxDOT has gradually introduced precast concrete components in bridge systems. For example, TxDOT already uses a number of precast superstructure elements, such as pretensioned girders with precast concrete deck panels. Even greater potential for economy and speed of construction is anticipated through the development of precast concrete bridge substructures. For example, bridge construction on large projects or over water crossings, which has proven to be slow and costly due to the large volume of cast-in-place concrete and associated formwork and field operations, can be substantially expedited.

The development of a precast bent cap system (Figures 1.1 and 1.2) is expected to be an important step in the advancement of precast substructures. Not only will such a system reap the benefits of reduced construction time and traffic disruption, but also allow the controlled conditions at precast plants to be exploited, enabling the efficient production of large numbers of high quality caps and facilitating the use of high performance concrete. In addition, a precast bent cap system could accommodate special construction conditions such as sites with difficult access or harsh environments more easily than cast-in-place systems.

Figure 1.1 Precast Bent Cap on Pierce Street Elevated Project

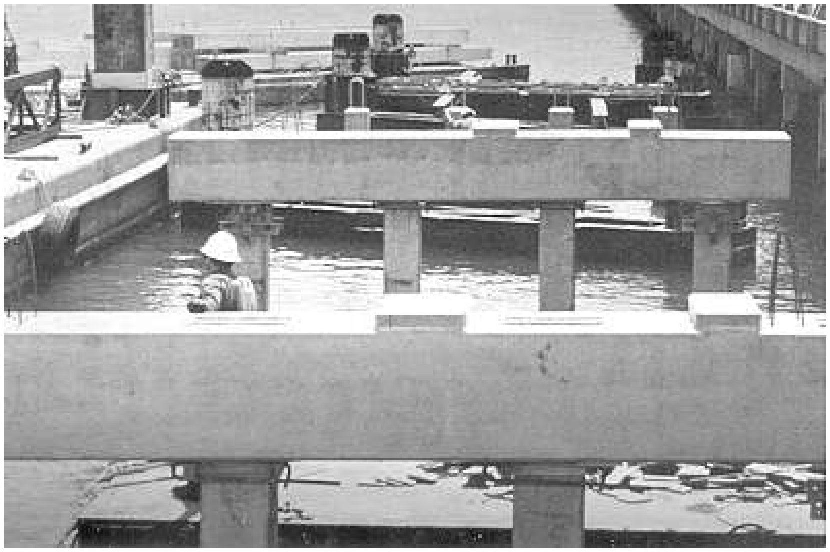


Figure 1.2 Precast Bent Caps on Red Fish Bay Project

1.1.2 Recent Precast Bent Cap Systems in Texas

Two recent TxDOT projects that used precast bent caps, the Pierce Street Elevated section of IH-45 in the Houston central business district, and the Red Fish Bay and Morris & Cummings Cut Bridges along the Gulf Intracoastal Waterway on State Highway 361 near Port Aransas, motivated TxDOT to initiate the formal development of a precast bent cap system. A brief review of these projects illustrates some of the issues that must be addressed in the development of a precast bent cap system.

1.1.2.1 Pierce Street Elevated

The Pierce Street Elevated project (PSE) involved replacement of 113 spans of the Pierce Street Elevated bridge superstructure and bent caps that were damaged by corrosion of reinforcement [1.1-1.3]. Figure 1.3 shows a segment of the PSE project during construction, and Figure 1.4 shows corrosion damage at the underside of a bent cap [1.4]. Existing columns were salvaged by saw-cutting column tops after removal of the bent cap. Rapid replacement was crucial because of the central location of the busy 1.6-mile segment. The original structure used cast-in-place inverted-tee bent caps on cast-in-place columns to support the pretensioned girders and deck. A replacement structure using similar construction methods would have required in excess of 1.5 years to complete. Because costs associated with user delays were estimated to be greater than $100,000 a day in urban Houston, TxDOT bridge designers decided to use precast bent caps to expedite construction. The use of precast bent caps allowed the Pierce Street Elevated project to be completed in a mere 95 days, less than 20 percent of the time estimated for conventional construction. As a reward for a 30-day reduction over the contractual agreement, the contractor received the largest incentive award ever paid in Texas—$1.6 million. Not surprisingly, the project was singled out by a major weekly newspaper in the 1997 reader's poll as the "Best Public Works Project" [1.3].

Figure 1.3 View of Pierce Street Elevated Project During Construction

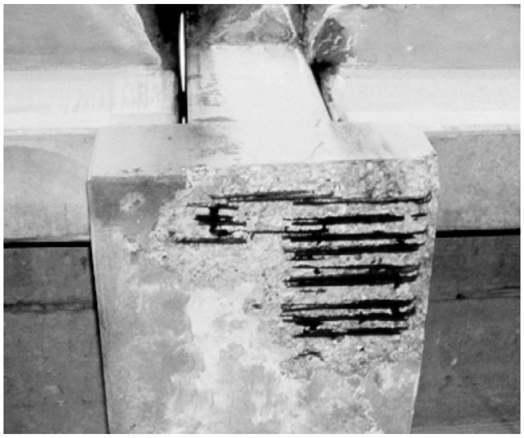


Figure 1.4 Corrosion of Original Pierce Street Elevated Bent Caps

The PSE project required the contractor to use construction operations that were somewhat unfamiliar and more complex than those used for traditional cast-in-place bents. To transfer forces between the inverted-tee cap and column, a bolted connection was designed and constructed. A rig was attached at the top of the columns for precise drilling of holes. High-strength Dywidag threaded bars were grouted into the holes, as shown in Figure 1.5. Select threaded bars were then proof-tested. A 150-ton crane was used to place the single-piece precast caps. Figure 1.6 demonstrates the procedure developed to "thread" the cap over dowel bars with the aid of small-diameter guide pipes of varied lengths extending through the cap depth. After a cap was set to the proper elevation using steel shims, the bedding layer at the cap-to-column interface was formed and cementitious, non-shrink grout (Euclid Hi-Flow) was pumped through grout ports (Figure 1.1) into the bedding layer between the cap and column top. The ducts encasing the Dywidag bars were then grouted and the bars tightened at the anchorage in the recesses at the top of the cap (Figure 1.7). Finally, recesses were grouted.

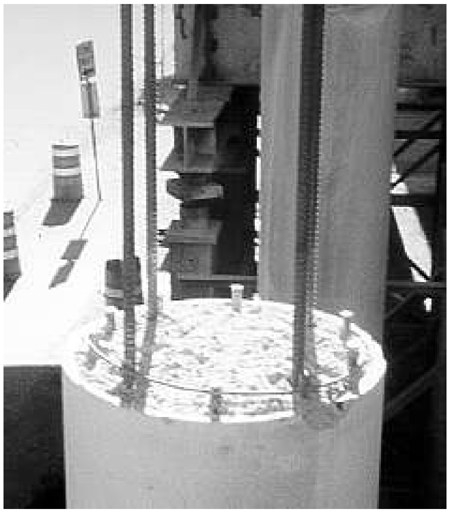


Figure 1.5 Dywidag Bars Grouted into Columns

Figure 1.6 Workers Prepared to Thread Dowel Bars through Precast Bent Cap with Aid of Extended Pipes

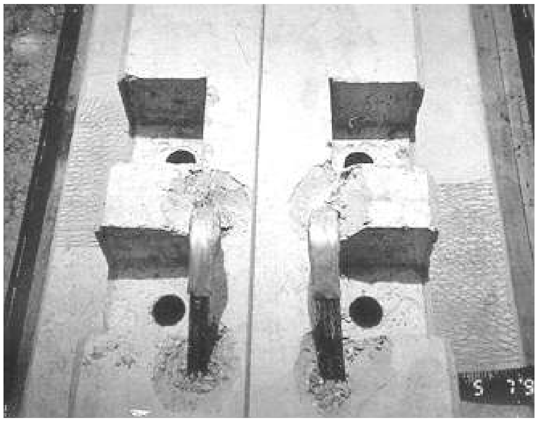


Figure 1.7 Recesses at Top of Bent Cap

1.1.2.2 Red Fish Bay

The Red Fish Bay and Morris & Cummings Cut Bridges (RFB) were designed and constructed as replacement structures because of severe salt water-induced deterioration in the original bridges [1.5]. RFB incorporated a pretensioned double-tee superstructure supported by precast rectangular bent caps on precast, prestressed 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 (Figure 1.2). Use of more than 60 precast caps greatly expedited construction operations, enabling construction to finish six months ahead of schedule. This was 1/3 shorter than the estimated construction time for cast-in-place bent caps. Durability protection was intended to be enhanced by the use of epoxy-coated bars, a non-shrink concrete with inorganic corrosion inhibitor, and a precast superstructure and substructure (with some elements prestressed). It was estimated that these measures will increase the life of the structure 50 percent at an initial cost increase of only 5 percent.

A modified version of a contractor-proposed grout pocket detail for connecting the precast cap to piles was used (Figure 1.8). The RFB construction sequence required driving piles using a steel frame as a template to satisfy horizontal tolerances. After piles were driven and cut back to the proper elevation, a double line of #9 U-shaped, epoxy-coated dowel bars was grouted with epoxy into embedded sleeves that had been cast in each pile. As shown in Figures 1.9 and 1.10, 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 support the cap. After adjusting cap elevations and forming the voids at the bottom of the cap, concrete with a maximum ¾-in. aggregate was cast in the grout pockets to complete the connection (Figure 1.11).

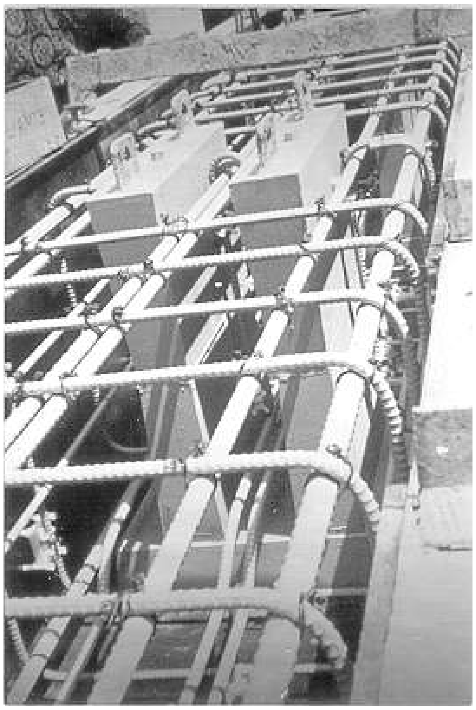


Figure 1.8 Red Fish Bay Grout Pocket Detail

Figure 1.9 Lowering of Precast Cap over Dowels

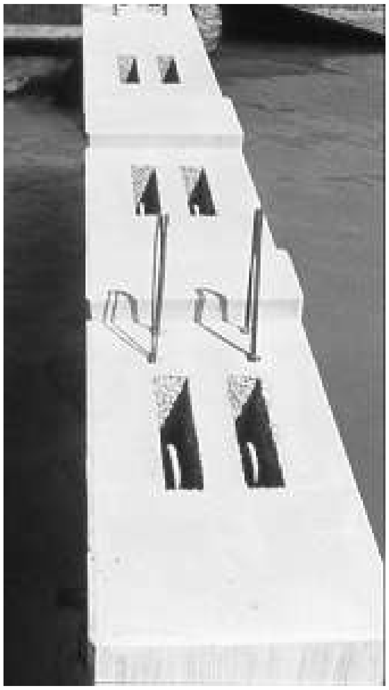


Figure 1.10 Grout Pockets After Cap Setting

Figure 1.11 Workers Casting Concrete into Grout Pocket

1.1.2.3 Challenges and Needs

The PSE and RFB projects convinced TxDOT engineers and contractors that the use of a precast bent cap system can significantly reduce construction time and cost. However, these projects also presented new challenges. For example, uncertainty existed regarding the design, detailing, behavior, and durability of such systems, especially regarding connections. In addition, new construction techniques have to be developed and mastered. More specifically, engineers and contractors identified the following specific needs for a precast bent cap system:

- A family of connection types and configurations
- A methodical approach for using existing products such as grout and connection hardware to design a cost-effective, constructible system
- A conservative design methodology for connections, including provisions for anchorage of bars within the connection
- Example connection details that satisfy design criteria and are compatible with conventional bent cap reinforcement details
- Specific construction guidelines, including alternative grouting approaches
- Approaches to mitigate potential durability problems related to exposed grout surfaces

1.2 OBJECTIVES

Realizing the potential benefits as well as uncertainties associated with a precast bent cap system, the Texas Department of Transportation, through the Center for Transportation Research (CTR) at The University of Texas at Austin, initiated CTR Project 0-1748, "Design and Detailing of a Precast Bent Cap System" [1.6]. The main objectives of the research project were to:

1. Develop practical, cost-effective candidate details for connecting precast bent caps to cast-in-place columns and precast trestle piles

2. Test select candidate connection details to examine connection constructability and behavior under loading

3. Develop a design methodology, example details, and construction guidelines for connecting precast bent caps to cast-in-place columns or precast trestle piles

A major emphasis of the objectives was to obtain "implementable" results.

1.3 APPROACH

1.3.1 Three-Phase Test Program

To accomplish the first objective, University of Texas researchers developed connection details in coordination with an Industry Review Committee (IRC) that included representatives of the precast and construction industries as well as TxDOT engineers. Discussions with the IRC helped in the development of four connection types: grout pockets, grouted vertical ducts, grouted sleeve couplers, and bolted connections. See Section 1.5 for a summary of design criteria related to economics, constructability, durability, and force transfer used in developing details. Discussions with the IRC revealed uncertainties in connection design and detailing, fabrication, and grouting. To address these uncertainties, University of Texas researchers conducted an experimental research program to investigate and refine grout pocket, grouted vertical duct, and bolted connection details. The test program included the following three phases:

- Phase 1—large-scale pullout tests examining the behavior and failure modes for grout pocket and grouted vertical duct connections (thirty-two tests)
- Phase 2—full-scale bent cap-to-column connection tests for grout pocket, grouted vertical duct and bolted connections (four tests)
- Phase 3—construction and testing of two full-scale bents at the construction yard of a highway contractor

Phase 1 pullout tests [1.7] were designed to investigate uncertainties associated with the anchorage of connectors embedded in grout pocket and grouted vertical duct connections, including the effect of variables such as connector anchorage (headed and straight bars), connector size, embedment depth, number of connectors, connector configuration, and confining reinforcement. In addition, uncertainties related to grout type (neat vs. extended), grout brand, grout strength, grouting operations, and interlock of grout pockets and ducts within a precast cap were examined. Because of the limited number of tests possible in the project duration, test specimens were designed for maximum comparison. Pullout tests were used as a simple yet reasonably conservative means to examine anchorage behavior and to provide a basis for developing connection details for Phase 2 tests.

Phase 2 investigated precast bent cap-to-column connection behavior through a full-scale beam-column test for each of the following connection types: single-line grout pocket, double-line grout pocket, grouted vertical duct, and bolted connection [1.8]. These tests served to verify the adequacy of representative connection details and to provide a further basis for the design methodology. Phase 2 assessed constructability, investigated the influence of the bedding layer on grouting and behavior, and determined the behavior of candidate connection details under service and factored loads, as well as at failure.

Phase 3 consisted of the construction and testing of a trestle pile bent and a column bent at the construction yard of a highway contractor, Champagne-Webber, Inc. near Houston, Texas. The main objective of Phase 3 was to assess the ease of construction for seven connections, including single-line and double-line grout pockets, grouted vertical ducts, and bolted connections. Connections were mixed within a bent to maximize contractor construction experience. Connection and bent behavior under loading were examined as a secondary objective.

1.3.2 Limitations on Scope of Research

For practical reasons, it was decided during the planning stages of the project that the scope of research be limited to conventional bridge systems that are predominant in Texas. Thus, the scope entails only precast bent cap on cast-in-place columns in multi-column bents or precast trestle piles. Bents subjected to seismic or other highly dynamic loads or bents of unusual proportions or applications are not covered. In addition, focus was placed on systems that use simple construction operations and provide maximum reasonable construction tolerances. Thus, post-tensioning of connections and bent caps as well as use of match casting, precast columns or other viable options were not addressed.

1.4 SCOPE

The following chapters and appendix document the accomplishment of the objectives. Chapter 2 discusses the development of candidate connection details in four categories: grout pockets, grouted vertical ducts, grouted sleeve couplers, and bolted connections. Advantages, disadvantages, and uncertainties are discussed, based on discussions with the IRC. In addition, issues associated with connectors and grout are addressed. Features of a grout specification and alternative grouting operations are introduced.

Chapters 3 through 5 document the three phases of experimental research, as explained in Section 1.3.1. Each chapter includes a summary of the test program, the lessons learned during construction operations, and test results. Chapter 3 summarizes thirty-two pullout tests conducted in Phase 1 on single-line and double-line grout pockets, as well as grouted vertical ducts. Chapter 4 presents the results of full-scale precast bent cap-to-column connection tests conducted in Phase 2 for each of the four following connections: single-line grout pocket, double-line grout pocket, grouted vertical duct, and bolted connection. Chapter 5 presents the results of Phase 3 construction and testing of a trestle pile bent and a column bent conducted at the construction yard of the contractor. Phase 3 centered on the construction of two bents incorporating seven connections, including single-line and double-line grout pockets, grouted vertical ducts, and bolted connections.

Using the results of Chapters 3 through 5, Chapter 6 summarizes the development of a design procedure for a precast bent cap system. A flow chart is used to illustrate the eight-step design procedure for the various connections. Equations for anchorage of straight or headed connectors in grout pockets or grouted vertical ducts are also developed based on Phase 1 test data.

Chapter 7 documents the development of a precast connection specification. Major components of a precast connection specification, including materials, a Precast Bent Cap Placement Plan, and grouting operations, are addressed based on the lessons learned during construction of the specimens in Phases 1-3.

Chapter 8 provides design recommendations and the precast connection specification, based on the developments of Chapters 6 and 7, respectively. In addition, example connection details that conform to the design recommendations and the precast connection specification are shown. Details include grout pocket, grouted vertical duct, and bolted connections.

Chapter 9 provides a summary and conclusion of the research, and lists areas for additional research. The appendix provides plan sheets for the first bridge designed using the design recommendations.

1.5 DESIGN CRITERIA

The various challenges and needs that must be addressed for successful development and implementation of a precast bent cap system relate to four design criteria: 1) economics, 2) constructability, 3) durability, and 4) force transfer. In this section, design criteria are discussed in general terms. See Chapter 2 for a specific discussion of these criteria in the development of candidate connection details. The following discussion is limited to a precast bent cap system that uses cast-in-place columns in a multi-column bent or precast trestle piles.

1.5.1 Economics

The development of precast bent cap systems is highly motivated by the potential economic advantage of precast construction [1.9]. In most cases, the greatest advantage of precast construction is the reduction in construction time and savings in time-related costs. As demonstrated by the PSE and RFB projects, time is saved because many time-consuming cast-in-place operations are avoided and some construction operations can occur in parallel. Reduction in construction time can also minimize risk and delays such as those due to flooding or wet weather.

Time savings is intricately linked to cost savings and both are highly dependent on the specific project. Time savings may be more important on small projects where even large percentage differences in cost per cubic yard of concrete will not result in a large total cost savings. For both large and small projects, construction documents include required completion dates and monetary penalties based on user costs for late completion. Because of the significant impact of PSE to the Houston central business district, a large incentive/disincentive clause was added to the contract. The rapid completion resulted in a large bonus. In urban areas, where bridges constitute a major portion of the work on a given roadway segment, efficiencies in bent cap construction can have a significant impact on the total project duration and cost.

Recently, a large Texas contractor estimated that a hypothetical project consisting of nine bents using inverted-tee caps could be completed in approximately 10 percent of the time required for equivalent cast-in-place construction, and at a direct cost savings of 30 percent [1.10]. For a job consisting of 25 bents using rectangular caps, it was estimated that the use of precast bent caps would require less than 20 percent of the time required for cast-in-place construction and result in a cost savings of over 20 percent. Unit costs for the precast products were developed after talking with producers and are conservative. Standardization will reduce time and cost, as economies of scale are expected due to form reuse, standard fabrication methods, and experience.

Life-cycle costs can also be reduced as a result of quality improvement. Quality is improved because work is performed at ground level, where ease of access and inspection result in greater attention given to the work. In addition, potential exists for fabrication of bent caps in the controlled environment of a precast plant, which also facilitates the use of high performance concrete.

Construction of a precast bent cap system requires more specialized construction expertise, tighter construction tolerances, and transportation, handling, and erection of potentially heavier elements (i.e., a single-piece bent cap). If not properly addressed, these three issues may offset the major advantages of a precast bent cap system. Implementation of any new system requires the construction community to gradually learn new construction methods. To implement a precast bent cap system, the contractor needs to develop specialized expertise. A higher level of specification is required for construction and grouting operations, as well as for the grout itself. Costs may be kept to a minimum by making construction and grouting operations as simple as possible, and by using prepackaged grout and readily available equipment. However, it will take time for the construction community to train workers and gain experience with erection procedures, such as placement of the cap and grouting of the connections.

It is anticipated that precast bent caps will be included in the plans as an alternate construction method. In this way, the contractor has more options, which will facilitate the evolution of a better product at a lower cost through refinement of details and procedures. The experience in Texas with precast panels for bridge decks is illustrative. For many years after precast deck panels were introduced, a cast-in-place option for bridge deck construction was shown in the plans. Recently, Texas has switched to standardized bridge slab reinforcing details, based primarily on precast panel bridge decks. This recognizes the nearly unanimous choice by contractors to use precast construction techniques in bridge deck construction. This plays a major role in the excellent cost efficiency of bridge construction in Texas. However, the preference for precast construction of bridge decks did not become apparent for many years. It took time for the idea to become accepted in the construction community and for precast facilities to perfect fabrication techniques. Showing alternate methods in the plans allowed for feedback from contractors and aided in perfecting standardized precast panel details. While great effort has been exerted to develop economical precast connection details (see Chapter 2), testing in the marketplace and refinement of standard details will be required before precast bent cap construction achieves its full potential.

Required construction tolerances can also offset cost advantages. Precast construction typically requires more accurate placement of components. Greater accuracy in column or pile placement requires that additional time and effort be dedicated to construction and thus increases the cost. Designing precast connections to be more “forgiving,” i.e., with more reasonable construction tolerances, can help mitigate such costs.

Costs may also increase if precast bent caps weigh significantly more than individual superstructure girders. Typically, a crane of a given size will be made available on the construction site to erect girders. On a small project, equipment costs are a significant portion of the total bridge cost. As an example, for a bridge using AASHTO Type IV girders up to 120 ft in length, a crane with a capacity of 60 or 80 tons may be used. If an 80-ton crane were used with rectangular caps weighing 1.6 kips/ft, the cap length would be limited to 100 ft or less. This would satisfy most standard roadway widths. On the other hand, inverted-tee caps may weigh between approximately 2.5 to 4.5 kips/ft, with the larger weights applying to caps supporting Type IV girders. In this case, the cap length may be limited to approximately 36 to 64 ft. Caps used for trestle pile bents would normally be significantly lighter than caps used with cast-in-place columns, although crane capacities may be more limited for water crossings if a barge draft is used or soft soils are present.

The crane capacity may have to be increased if the bent cap is much heavier than the beams. Crane capacities are more limited in some areas of Texas and can also be limited by site conditions. In some cases, long bents can be constructed as a series of independent shorter bents to control cap weight. However, designers need to decide if this unduly detracts from aesthetics or increases the cost over cast-in-place bents due to the requirement for additional columns and foundations or piles. Bent caps can also be precast in smaller segments and post-tensioned in the field [1.11]. This approach could reduce the segment weight and enhance durability, but would increase the level of skill required for fabrication and construction. Shipping costs for the bent cap, if fabricated off-site, may also increase significantly. Impacts of cap weight and size can be determined by communicating with highway contractors experienced in a given area.

1.5.2 Constructability

To maximize construction efficiency and cost-effectiveness, a precast bent cap system must be relatively easy to build. Constructability issues apply to both fabrication and erection. The previous section already discussed various constructability issues, as constructability is inexorably tied to the economics of the system. A precast bent cap system would most likely be used in a multi-column cast-in-place bent or a precast trestle pile bent. The use of a precast cap with cast-in-place columns allows column casting to follow that of the foundation, as is normally done. However, the use of a precast cap avoids the requirement for bent cap formwork and casting on site. Much of the superstructure would likely be precast.

A precast bent cap system using precast trestle piles would appear to have a significant advantage of using an entirely precast substructure. However, the effort required to maintain pile-driving tolerances offsets some of the perceived advantage. As mentioned for the RFB project, a steel frame was required to achieve pile placement within tolerances. Nevertheless, construction still can proceed much more quickly than for cast-in-place construction.

Efficient construction relies heavily on the connection type, construction sequence, and the use of a suitable grout and grouting operations to complete the connection in the field. Various options exist for connection hardware, prepackaged grouts, and grouting operations. Cap setting operations require appropriate use of shims, friction collars, or shoring, together with geometry control to provide for the cross slope. Gravity-flow grouting or pumping may be used to grout connections.

1.5.3 Durability

Durability is often a major concern for bridge substructures in Texas. The PSE and RFB projects were initiated due to corrosion damage. Depending on connection type, the use of a precast bent cap system may pose new durability concerns not present for a cast-in-place bent. For example, RFB used grout pocket connections, which may pose uncertainties related to shrinkage of grout or concrete in the pockets and durability resistance of exposed surfaces. Durability concerns for PSE connections include exposed surfaces at the bedding layer and recesses, and potential opening at the bedding layer. Moisture paths into the connection due to air entrapment are always a concern for grouted connections.

Connections can be designed to enhance durability. Designers can select connection types and configurations that minimize exposed surfaces, and exposed grout surfaces can be minimized and sealed. Bent cap reinforcing bars can be spaced to minimize service level cracking in the connection region and high performance concrete can be used for the precast cap. Grouts that are resistant to corrosion or degradation can be used. Grouts can also be tested for suitable workability to ensure a well-consolidated connection. Connectors can be epoxy-coated and use large cover, and ducts and other connection hardware can be galvanized or otherwise protected.

1.5.4 Force Transfer

1.5.4.1 Connection Classification

The designer of a precast bent cap system must know how the connections and overall structure will behave. Based on this understanding, the designer can realistically determine the design actions at the limit states and design and detail the connection (and structure) appropriately. Clearly, the major differences between precast and cast-in-place connections need to be considered for analysis and design.

Stanton and others have reviewed and classified precast connections [1.12,1.13,1.14]. Precast connections are classified as either “wet” or “dry.” Wet connections are cast-in-place connections used to join precast elements. The intended result is that the precast structure emulates, i.e., behaves the same as, a cast-in-place structure. In this way, a framing system is made tough and ductile against all load effects regardless of whether such response is necessary. This approach has been widely used in the seismically active regions of the world, including Japan and New Zealand. Such an approach is similarly feasible in the U.S. but not often used because of the mixing of wet and dry trades and the complications associated with liability [1.12].

Precast construction in the U.S. normally uses dry connections. This normally includes connections that use grout as well as mechanical connections with steel. Dry connections usually form natural discontinuities in strength and stiffness. In the absence of special provisions, this will lead to a concentration of inelastic action at the joint. For seismic connections, this may reduce the seismic energy delivered to the structure but may also result in inadequate energy dissipation. However, for nonseismic and low seismic regions such as Texas, concentration of inelastic action at a joint may be acceptable. In addition, dry-jointed connections can have the advantage of achieving the desired moment resistance at a larger deformation and rotation at the connection, which can act to relieve potential stress buildup in the system due to shrinkage or changes in temperature.

Dry connections have been classified in three performance categories [1.12,1.13]: strong, deformable, and energy-dissipating. A strong, or rigid, connection is designed with a greater ratio of strength to applied force than the adjacent member so that the connection never yields. This is not necessarily easy or desirable. Deformable and energy-dissipating connections are considered ductile connections. A deformable, or extensible, connection (e.g., elastomeric bearing pad) can undergo significant deformations in one sense without endangering its integrity in another. An energy-dissipating connection for a nonseismic region has sufficient inelastic deformation capacity to undergo the imposed strain history.

1.5.4.2 Connection Testing

Stanton reported that a major divergence of opinion exists over the best connection detail for a given situation [1.12]. Significant differences of opinion exist for various regions of the country as well as professions. For example, a designer may desire a connection to be strong and ductile, but a fabricator and contractor would like ease in fabrication and erection. Accommodating all interests sufficiently is often difficult. As mentioned previously, the economic advantage of a precast bent cap system depends greatly on constructability.

Connection details are developed based primarily on constructability and economic considerations (see Chapter 2). However, because the integrity of the system depends primarily on the connection performance, representative details should be constructed and tested. This will ensure a proper understanding of structural behavior and thus help develop a conservative approach for analysis and design. If a precast bent cap system were to use the same number of connectors as is typically used for cast-in-place bents (e.g., extending 8, 10, or 12 column bars into the cap), then the primary differences in structural performance for a precast system would likely be related to the presence of a grouted bedding layer, anchorage of the connectors in a grouted connection, and potential embedment of the column or pile into the cap. The existence of a cold joint at the column top is not a difference between cast-in-place and precast systems, as a cold joint would exist for both cases. Such an approach may indeed rest in similar performance to a cast-in-place system. However, it is possible that fewer connectors may be used for a precast bent cap connection (e.g., 4, 6, or 8 connectors). Such an approach may not only facilitate construction, but also provide adequate force transfer. It is prudent, however, to verify connection strength, stiffness, and ductility by testing. This will also help determine if a new analytical approach is required for design. Thus, the design strategy for a precast bent cap system is to develop a constructible, economical and durable system that provides the requisite performance at the various limit states, but not necessarily emulate a monolithic system.

If testing reveals that inelastic action concentrates at the beam-to-column interface, then the connections may be considered dry-jointed. Such connections will likely be ductile, energy-dissipating connections because producing a rigid connection will likely offset the economic and constructability advantages of a precast bent cap system. Testing will also help determine the extent of the inelastic action and verify if a desirable level of ductility is achieved. Additionally, it is essential that development length requirements for connectors be established through testing. Finally, testing can also help establish joint confinement requirements.

1.5.4.3 Preliminary Analysis

A series of analyses were performed to determine a range of connection forces for multi-column cast-in-place and trestle-pile bents using rectangular and inverted-tee caps [1.15]. This was intended to help define loads and load eccentricities (i.e., moment/axial load) at the column top for developing realistic connections and test specimens. Load combinations specified by the AASHTO Standard Specifications and AASHTO LRFD Bridge Design Specifications were used [1.16,1.17]. Connection actions, including design moments in the plane of the bent (transverse direction) and out-of-plane (longitudinal direction), were determined. Combined longitudinal and transverse (simultaneous) eccentricities were also considered. Load levels and eccentricities are discussed further in Chapter 4.

1.6 LITERATURE REVIEW

The literature review is divided into the following three areas: 1) precast substructures, 2) precast connections, and 3) anchorage. Chapters 2 and 6 provide a further discussion of some references and introduce additional references.

1.6.1 Precast Substructures

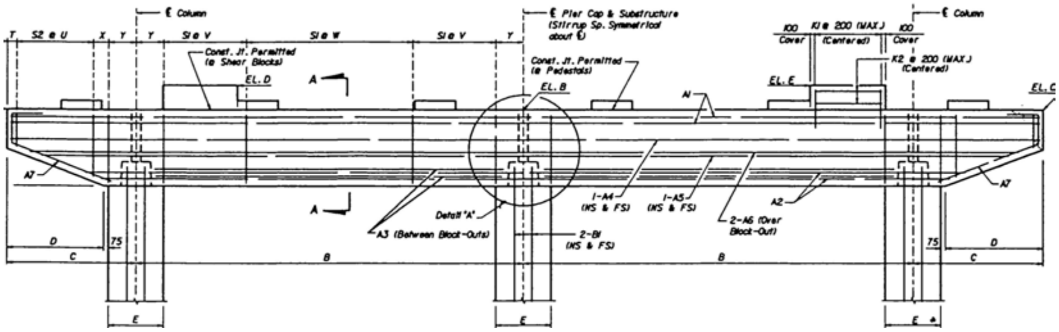
In recent decades, a number of noteworthy projects have used precast substructures, such as the Linn Cove Viaduct in North Carolina (1983), the Chesapeake and Delaware Canal Bridge in Delaware (1995), and the US 183 in Austin, Texas (1997) [1.18,1.19,1.20]. Work has also been conducted to develop precast substructures for standard bridges [1.21]. The focus of those efforts was mostly on segmental construction and post-tensioned connections, which are not addressed by the current research. The following projects are among the few known cases documented in the literature that incorporated precast bent caps.

1.6.1.1 FDOT

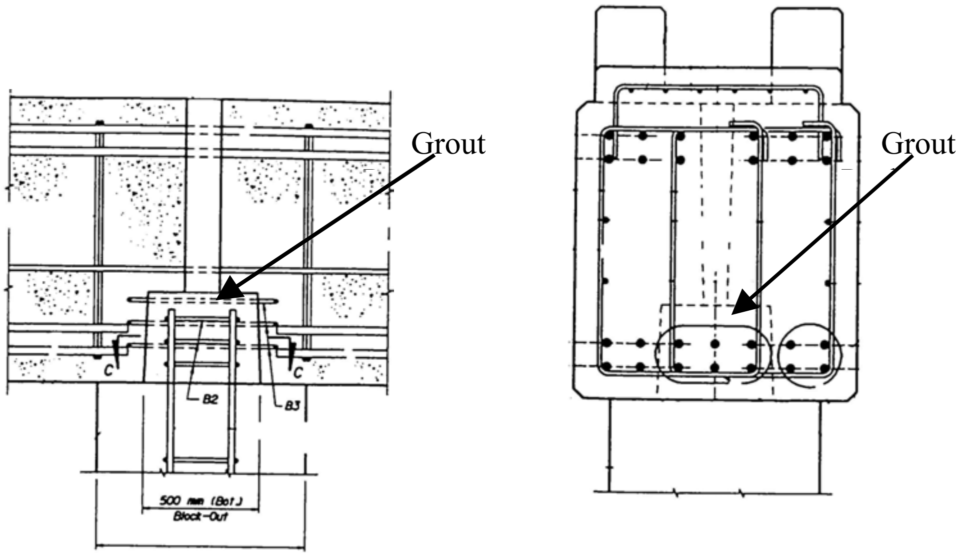
The Florida Department of Transportation (FDOT) and its primary contractor, LoBouno, Armstrong, and Associates (LAA), recently completed an effort to develop a family of standardized shapes and connection details for precast substructures [1.22]. FDOT is considering use of precast substructures in routine, moderate span bridges for both water and grade crossings. Based on nationwide DOT and industry surveys and evaluation criteria, FDOT investigated numerous bent cap and column configurations for typical applications including precast trestle pile bents and single-column (hammerhead) and multi-column bents. Mechanical couplers, post-tensioning, welded connections, and grout pockets were considered as connection alternatives, although connection details focused on grouted sleeve couplers and pockets filled with concrete or grout. For multi-column bents, precast bent cap cross sections selected for further development included solid rectangular and inverted-U shapes on hollow rectangular or H-shaped precast columns.

Figures 1.12 and 1.13 show draft semi-standard drawings for column and pile bents [1.23]. Figure 1.12 shows a rectangular cap using a single-line grout pocket connection for a multiple column bent.

Connections are filled with concrete or grout from an opening at the top of the cap. Four connectors are shown shallowly embedded into the cap in a confined region. Some longitudinal reinforcement is discontinued at the grout pocket. No bedding layer is shown. An alternative connection using grouted sleeve couplers was also developed. For grade crossings, the weight of elements is limited to 60 tons for handling purposes. Figure 1.13 shows a rectangular cap used with piles. The section view shows a large void formed in the cap to house the pile. Large tolerances are accommodated with such a detail. Connection voids are filled from two holes that extend through the cap depth. No positive connection is shown. An alternative detail showed a slight modification to the pocket shape to provide for battered piles. An industry review board evaluated the developed details. No testing of details was conducted.



A. Elevation



B. Close-up of Connection Detail

C. Cross Section of Cap

Figure 1.12 Florida DOT Rectangular Precast Bent Cap for Column Bent [1.23]

Reference 1.24 summarizes additional developments in FDOT's precast substructure effort, including criteria, design procedures, semi-standard drawings, and special technical provisions. Final products have not yet been released. Precast pile caps were developed, in addition to precast caps for both single-column and multi-column bents. Weight of precast pile caps is limited to 40 tons and standard cap widths were developed to accommodate 18-in. and 24-in. prestressed piles. Connections are made in a manner similar to that shown in Figure 1.13. No connectors are used between the cap and pile. Grout pockets are sized to provide a horizontal tolerance of +/-4.5 in. Design accounts for potential loss of a pile.

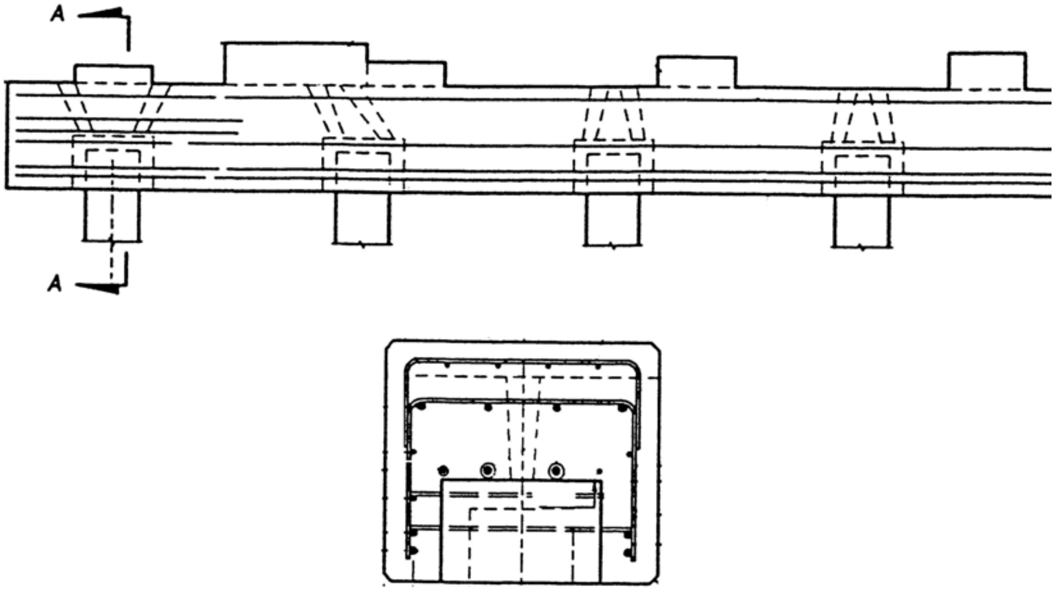


Figure 1.13 Florida DOT Rectangular Precast Bent Cap for Pile Bent [1.23]

Multi-column rectangular bent caps with depths between 3 ft and 4.5 ft were developed, based on span lengths of approximately 80 to 160 ft. Weight of precast caps is limited to 60 tons. Connections were developed assuming the beam-to-column joint is pinned in the transverse (in-plane) direction, but provides sufficient moment capacity in the longitudinal (out-of-plane) direction so the cap does not roll off the columns. A positive connection is made by extending reinforcing steel from the columns into the cap (Figure 1.12). Grouted sleeve couplers were proposed for cases requiring moment resistance in both directions.

For two-column bents, grouted sleeve couplers were selected over post-tensioning systems based on the successful application of grouted sleeve couplers in the Edison Bridge, and the fact that more manufacturers of grouted sleeve couplers will make this option more cost-competitive. Tight quality control for grouted sleeve couplers was highlighted. Close coordination between the precaster and contractor was considered indispensable, and it was recommended that one manufacturer make templates for use by both the precaster and contractor. Column reinforcement of 1 percent of the gross area for H-shape columns was expected for grouted sleeve coupler connections. However, it was noted that the AASHTO Standard Specifications allows for smaller than 1-percent reinforcement if the cross section is larger than required and that AASHTO LRFD Bridge Design Specifications allows column reinforcement as small as 0.7 percent. A minimum number of connectors was recognized as advantageous. Caps as large as 80 tons were considered. Design followed a process similar to that used for cast-in-place bents.

1.6.1.2 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 [1.25]. Precast bents consisting of inverted-U bent caps and I-shaped columns were chosen to minimize weight (Figure 1.14). Bent cap lengths were limited to 61 ft. and approximately 80 tons. Columns did not exceed 41 ft. and weighed less than 50 tons. 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, precast caps were placed level to ensure successful placement over dowels. A cross slope of two percent was achieved by varying pedestal heights.

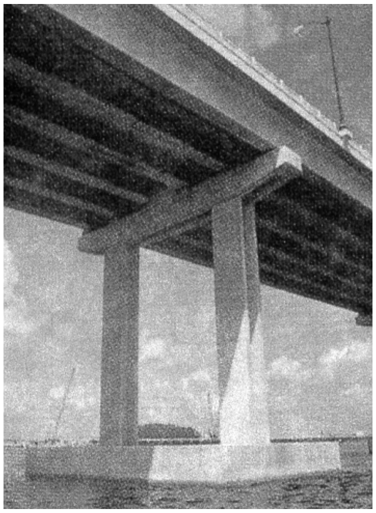


Figure 1.14 Precast Inverted-U Bent Caps on Precast I-Shaped Columns on Edison Bridge [1.25]

1.6.1.3 Getty Museum People-Mover System

The use of precast crossheads (i.e., bent caps) on single-column or two-column bents was critical to the success of the 3535-ft long tram guideway for the Getty Museum people-mover system in Southern California [1.26]. The key to successful implementation of the precast caps was the use of a special reinforcing bar template on the top of the 42-in. diameter columns. This template guaranteed proper alignment of the 16 vertical dowels that extended from the cast-in-place columns into corrugated sleeves embedded in the cap (Figure 1.15). Due to placement of cap reinforcement, sleeves were only 1.5 in. in diameter, requiring an extremely tight tolerance and leaving a small clearance for dowel bars that were as large as #11's (1.41 in. nominal diameter). A fabricator used a single, precise jig for both field and shop templates to prevent error in dimensions. In addition, templates were used in the base plate and top of the cap formwork to precisely position the sleeves. All caps fit and aligned accurately in the field. A 180-ton truck crane was used for erection, although caps weighed no more than 40 tons. After setting the cap, sleeves were grouted with high-strength grout. A full-scale mock-up constructed in the precast yard prior to production was deemed a major contribution.

1.6.1.4 Texas State Railroad Bridges

A precast alternate was developed for the Texas State Railroad Bridges using precast caps with driven steel H-piles for support of precast box beams [1.27]. An isometric view of this system is shown in Figure 1.16. While the construction sequence is similar to the sequence used in RFB, 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 to be embedded into the cap approximately half the cap depth. To facilitate bearing, a steel plate welded to the top of the piles was proposed. Two grout ports extended through the cap, similar to the FDOT detail for precast piles. This concept was not implemented because the contractor opted to use an alternate design.

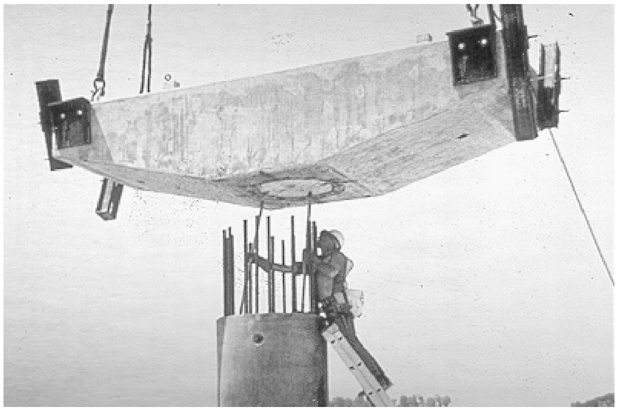


Figure 1.15 Threading of Precast Cross Head over Extended Column Bars on Getty People-Mover Guideway [1.26]

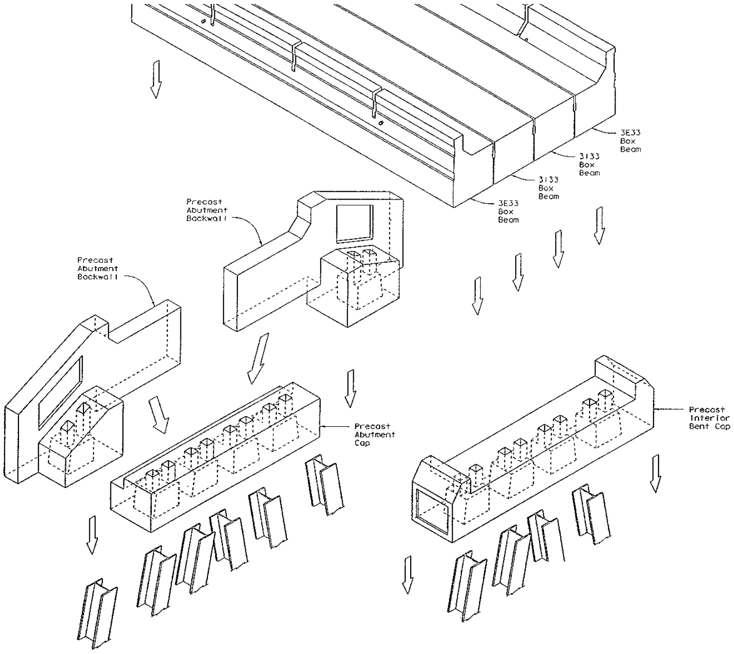


Figure 1.16 Isometric of Texas Railroad Precast System [1.27]

1.6.2 Precast Connections

The previous section included a discussion of connections for precast bridge substructures. The majority of published literature on precast connections, however, centers on building construction, particularly in seismic regions. A review of pertinent concepts related to both construction and force transfer is included below.

1.6.2.1 Types

As mentioned in Section 1.5.4, Stanton and others reviewed and classified precast connections. Although valuable principles are included in their discussions, most of the actual connection types do not apply to a precast bent cap system. Martin and Korkosz [1.28] and the Precast/Prestressed Concrete Institute (PCI)

Connection Manual [1.29] recommend approximately two dozen precast beam-to-column connections for building construction, including simple and moment-resistant connections that are bolted, doweled, or welded. Although most connections do not use a continuous beam above a column, some connection types are applicable to a precast bent cap system. Martin and Korkosz evaluated connections on the basis of simplicity, durability, and accommodation of volume change. It was assumed that connections could be designed adequately for strength and ductility. Bolted connections provide oversized holes for adequate tolerance, easy erection in any weather, recesses for corrosion protection, potential post-tensioning, and moment resistance. Doweled connections use sleeves that are grouted after placement of the beam. These can be concealed and in some cases can be sized to accommodate reasonable tolerances. Welded connections are not preferable due to uncertainties in quality control, inspection, and possible embrittlement of bars or welds. Full moment resistance is provided in some instances by the use of cast-in-place concrete in the joint region. Connection types and construction sequences are further discussed in Chapter 2.

1.6.2.2 Grouting

1.6.2.2.1 New Zealand Concrete Society

A study group of the New Zealand Concrete Society and New Zealand National Society for Earthquake Engineering provided an excellent summary of key aspects of grouting [1.30]. Cement-based grouts should provide a strength greater than the surrounding concrete. Grouts should be low viscosity (i.e., fluid) to properly fill voids and prevent entrapped air. In vertical void grouting, corrugated metal ducts rather than PVC or other materials should be used to ensure adequate bond. Drilling of vertical holes should be conducted in such a way that grout-concrete bond strength is not reduced by micro-fractures, polishing, or residual debris. Proper sealing of forms is essential. Grouting operations are specialized practices that require a high standard of workmanship. Gravity-flow grouting by tremie-tube grouting or decanting is suggested (see Section 2.4.2.1 for detail). Horizontal voids require air venting. Grouting of beam-column connections and a bedding layer using vertical ducts can be accomplished by pumping grout from the bedding layer upward or by grouting with a tremie tube down a corner duct. Due to the difficulty in assessing grouting quality after grouting operations, greater emphasis must be placed on inspection during grouting operations. Pullout tests may be necessary to verify installation.

1.6.2.2.2 University of Canterbury

Restrepo reported grouting operations as part of a large test program investigating seismic behavior of connections between precast concrete elements [1.31]. One system developed for beam-column connections used corrugated steel ducts similar to those used in post-tensioning within the beam. After the precast beams were set on steel shims, the bedding layer with a maximum thickness of 1 in. was formed and grouted together with the ducts. Column dowels extended through the beam for splicing or coupling at the next level.

Trial batches of three cement-based grouts were investigated. One was a custom mix developed by the university's concrete laboratory and the other two were proprietary prepackaged mixes—Sika 212 and Monogrout. A grout flow (i.e., efflux time based on a standard flow cone measurement), of 40-50 seconds was targeted to allow grout to flow easily through the bedding layer while avoiding excessive segregation. In addition, grout strength based on 2 in. x 4 in. cylinders was intended to be at least 1.4 ksi greater than the surrounding concrete in the precast unit. Sika 212 exhibited a flow of over 60 seconds for both batches, with some segregation. Monogrout achieved a flow as low as 17 seconds, and exhibited significant segregation, plastic settlement, and bleeding. Grout compressive strengths were determined from samples taken from dummy corrugated ducts (2.6-in. diameter, 30-gage) that were grouted and sliced for testing.

After presoaking the bedding layer for six hours, an unsuccessful attempt was made to grout the connection using the custom batch of grout. The grout segregated and packed within the grout pump and hose. Gravity-flow grouting was then carried out using a tremie tube. Topping off of ducts was needed due to loss of hydraulic head. This was actually considered advantageous in making the grout more homogeneous through the depth, as it appeared that more of the coarser material remained at the bottom of the ducts and bedding layer. The top 2 in. of grout was removed due to excessive bleeding. It was recommended that alternative grouts that do not exhibit segregation be evaluated.

1.6.2.2.3 Guide Specification for Grouting of Post-Tensioned Structures

The Post-Tensioning Institute (PTI) Committee on Grouting Specifications has developed draft guidelines for grouting of post-tensioned structures [1.54]. Grout and duct materials, grout design, quality control for materials, and construction are addressed. Although focus is placed on custom grout mixes rather than prepackaged grout, valuable guidelines related to setting time, compressive strength, permeability, volume change, flow, bleeding, and an accelerated corrosion test are provided. In addition, practical guidelines for grouting operations and trial batches are given.

1.6.2.3 Tests

1.6.2.3.1 PCI—Grouted Ducts in Beam

Stanton et. al. reported test results and a design approach for eight moment-resistant beam-column connections [1.32]. Two moment connections used dowels that were either partially or fully grouted in ducts (Figure 1.17). Specimens were loaded monotonically as shown in Figure 1.18. Both specimens exhibited very ductile response. Dowels embedded approximately 24 times the bar diameter ($24d_b$) achieved yield, and specimens reached a tip rotation of at least 4 percent. Failure for the fully-grouted specimen occurred when the bar fractured, while testing was stopped after a large rotation of the other specimen. Debonding along a portion of the dowel did not improve performance. Cyclic loading on one specimen indicated little energy dissipation. Moment capacities were larger than predicted, but still fairly small due to the small number of bars (four) and small bar size (#6).

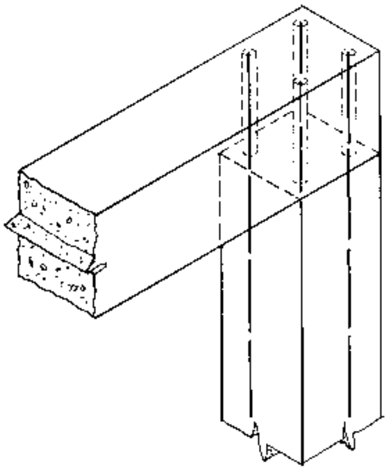


Figure 1.17 Schematic of Grouted Duct Connection [1.32]

Due to uncertainty in the effectiveness of increasing the size or number of dowels, these connection types were recommended for cases where a small moment transfer is required, such as an interior column. Concern was expressed regarding anchorage of reinforcing bars in relatively shallow beam depths if full-moment transfer is desired. Proper confinement details were recommended to prevent excess cracking and spalling at the edges of the beam. Fabrication and erection were deemed simple. Dowel alignment was considered critical, and curing time for grout was noted as a delay.

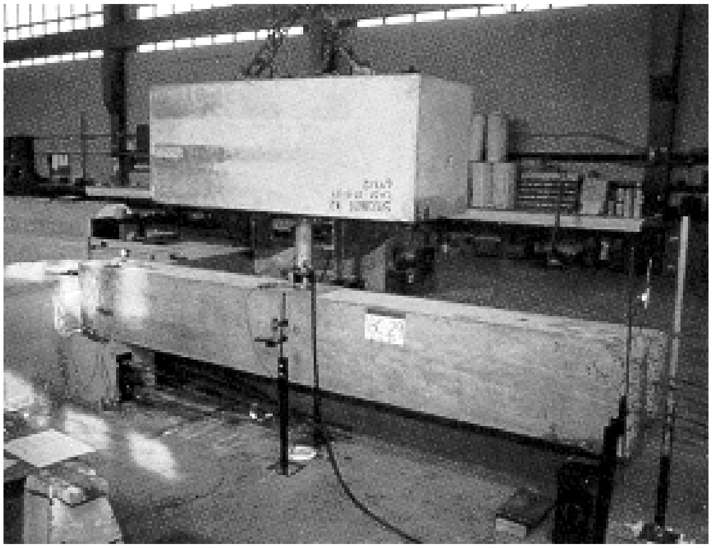


Figure 1.18 Testing of Grouted Duct Connection [1.32]

Figure 1.19 shows five failure mechanisms for this connection: 1) yielding of beam rebar if sufficient moment transfer develops; 2) yielding of dowels near the bedding layer, resulting in opening of a gap at the joint; 3) crushing of concrete and grout at the edge of the column, with possible contribution due to volume restraint; 4) bond failure and pullout of bars; and 5) yielding of reinforcement along a debonded length. Dowel yielding is considered the most ductile failure mode, if beam bars do not yield.

The recommended design approach for connection design includes the following:

1. Analyze the structure assuming a fixed connection. Redistribute moments as much as possible, and conservatively size positive moment reinforcement at midspan.
2. Design beam reinforcement and dowels for the design moment based on a fixed connection. Conservatively size dowels so that beam top bars at the connection yield first.
3. Check development lengths of dowels in the beam. Use additional dowels to compensate for incomplete development of capacities if full-moment capacity is needed. Otherwise, do not rely on the connection for full moment capacity and provide full positive moment capacity at midspan.
4. Provide anchorage for dowels within the column.
5. Provide column confining reinforcement for moment and tension due to volume restraint.

It is recommended that the structure be analyzed for a reduced moment capacity at the connection and moment redistribution in the structure if the connection is considered to provide partial moment resistance.

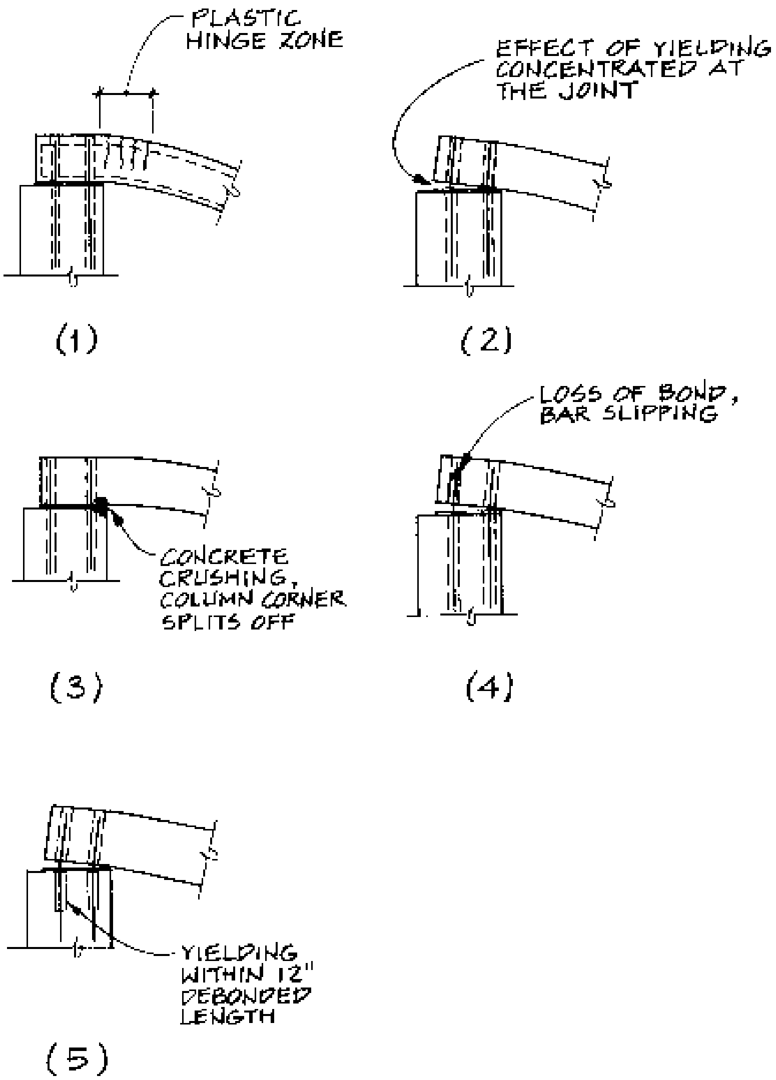


Figure 1.19 Failure Modes for Grouted Duct Connection [1.32]

1.6.2.3.2 Imperial College—Grouted Ducts in Column

Zheng reported the behavior of eight doweled and grouted precast concrete column connections [1.33]. Although the specimen was subjected to reversed cyclic loading combined with axial compression, results have bearing on a precast bent cap system. The connection was formed by grouting four #5 dowel bars that extended from a top stub column a distance of approximately $50d_b$ into 2-in. ducts in a lower stub column. Variables included the thickness of the bedding layer and lateral reinforcement in the joint region. Ducts were formed using PVC that was removed shortly after casting. The bedding layer was grouted using pumped grout. Bleed holes were incorporated to prevent air entrapment. For cases in which the load was applied at a large eccentricity, failure was initially characterized by the formation of a horizontal crack at the joint interface. This was followed by the formation of vertical cracks and spalling in the concrete above and below the joint on the compression side. For small eccentricities, horizontal cracking did not appear. The monolithic specimen appeared more ductile at failure due to a larger failure zone that extended beyond the immediate joint vicinity. However, capacity for all tests was predicted closely using an axial load-moment (P-M) interaction curve based on conventional reinforced concrete theory. Specimens with a 1-in. bedding layer exhibited a smaller strength reduction due to load reversals compared to specimens with a 2-in. bedding layer. The effect of tighter spacing of lateral reinforcement in the joint region was inconclusive.

1.6.2.3.3 New Zealand—Grouted Ducts in Building System

In addition to the grouting operations discussed in Section 1.6.2.2.2, Restrepo reported test results at the University of Canterbury for beam-column connections of grouted column bars that extend through corrugated ducts [1.31]. Excellent stiffness and ductility of the system were exhibited, despite segregation during grouting. In addition, the corrugated ducts, grout, and bedding layer performed very well, with no significant difference in behavior from monolithic construction. The use of grout strength at least 1.4 ksi higher was considered satisfactory. The same overall conclusions were made for prior tests on the same frame system that used a grout with a strength comparable to the surrounding concrete. Park reinforced these conclusions in a more recent article [1.34].

Appendix B of Reference 1.30 reports that tests at Works Central Laboratories compared the pullout behavior and bond strength of grouted vs. cast-in-place column bars. It was found that grouted bars had a slightly larger pullout strength, which was attributed to the strength of the grout being higher than the surrounding concrete. Failure occurred due to breakdown of bond between the duct and precast concrete rather than bond between the bar and grout. Some bars reached yield at lengths much less than code development lengths. Monotonic and cyclic pullout tests were also conducted on specimens using confining ties around the ducts. Ties yielded in all cases, and little difference was found between the cast-in-place and grouted specimens. It was concluded that grouted ducts could be used with confidence. Because these tests used a grout compressive strength at least 1.4 ksi greater than that of the precast concrete, it was recommended that future applications require this margin.

1.6.2.3.4 University of Nebraska—Generic Grouted Sleeves

As an alternative to proprietary grouted sleeve couplers, Einea et. al. tested 14 generic grout-filled steel pipe splices in monotonic tension to failure [1.35]. Bars extending from the splice sleeve were used for tension testing without embedding the specimen in concrete. No. 5 to #9 bars were embedded between approximately 5 d_b to 11d_b using standard steel pipes as large as 3 in. in diameter. High bond strength was achieved by confining the high-strength grout around the bars. Confinement prevented grout splitting so that failure was characterized by shearing of the grout at the bar lugs. Bars were developed in lengths as short as 7d_b. However, more extensive tests were recommended to investigate the effect of grout strength, pipe geometry, bar slippage, and cyclic and fatigue loading. In addition, anchorage of the sleeve within concrete was not addressed.

1.6.2.3.5 NMB Grouted Sleeve Coupler

Testing of proprietary grouted sleeve couplers is normally unavailable. However, NMB has reported that when their coupler is used together with high-quality bedding materials, structural behavior can emulate monolithic cast-in-place structures and thus can be designed according to code requirements for ordinary reinforced concrete [1.36].

1.6.3 Anchorage

This section summarizes relevant literature on anchorage of straight and headed bars in cement-based grout, as potentially applicable to Phase 1 pullout tests on grout pocket and grouted vertical duct specimens. A few references for anchorage of straight and headed bars in concrete are also discussed as a means of comparison with cast-in-place concrete anchorage. Chapter 6 discusses these references in further detail.

1.6.3.1 Bars Anchored in Grout

1.6.3.1.1 University of Kansas—Straight Reinforcing Bars in Drilled Holes

Darwin et. al. [1.37] reported the bond strength of individual reinforcing bars grouted in small-diameter holes drilled in concrete for repair and retrofit applications. The experimental program consisted of several hundred pullout tests on specimens that investigated the following variables: #5 and #8 reinforcing bars, embedment of approximately $6d_b$ to $19d_b$ in holes typically 0.25 in. larger than the bar diameter, uncoated and epoxy-coated bars, and six types of grout, including two cement-based grouts. A cover of 3 in. was used in most cases to represent typical bridge construction. Primary failure modes included: 1) pullout, 2) failure at the grout-concrete interface, accompanied by the formation of a shallow concrete cone, 3) splitting failure, 4) failure due to tensile and flexural cracks, and 5) a combination of the various modes. It was found that bond strength, defined as the maximum applied load for monotonic tension in a pullout test, increased with increasing embedment length, bar size, and cover. The effect of epoxy coating on bond strength was considered insignificant. The following best-fit equation was the basis for a proposed design equation in cases where a higher strength grout is used:

$$T = 30 l_e \sqrt{f'_c} \qquad (1\text{-}1)$$

where: T=bond strength, lbs

l_e=embedded length of grouted reinforcement, in.

f'_c=specified compressive strength of the concrete, psi

Bond strength was found to increase with embedment depth and the square root of the concrete compressive strength, as long as the grout strength was high enough to prevent failure at the grout-concrete interface. Applying a strength reduction factor of 0.65 to the bond strength, Reference 1.37 showed that this equation required a development length 22 percent less than that required by the American Concrete Institute (ACI) Committee 318-89 Building Code for #11 bars or smaller [1.38].

1.6.3.1.2 University of Florida—Straight and Headed Threaded Rods in Drilled Holes

Cook et. al. [1.39] reported pullout test results from a program developed to determine the strength and behavior of grouted anchors. Fifteen pullout tests (each repeated five times) were conducted on individual anchors grouted in 2-in. diameter holes drilled in concrete. Anchors included: 1) threaded rods, 2) threaded rods with heavy hex nuts, 3) threaded rods with hex nuts, and 4) a #4 reinforcing bar. Masterflow 928 was among the grouts used. For nearly all tests, embedment depths were between $6d_b$ and $7d_b$, with bar diameters from 0.625 in. to 1 in. Large edge distances were used to prevent splitting failure. All unheaded anchors exhibited bond failure at the grout-anchor interface, accompanied by the formation of a secondary shallow cone in the concrete. A uniform bond stress model for unheaded anchors was established for design based on test values of bond strength for each brand of grout. Headed bars grouted with MF928 exhibited a full concrete cone failure similar to that formed in cast-in-place concrete. Anchorage strength for headed bars was closely and conservatively predicted (within 13 percent) using the Concrete Capacity Design Method [1.40, 1.41], described in Section 1.6.3.2.2.

1.6.3.1.3 PCI—Straight Reinforcing Bars in Ducts

The PCI Design Handbook reports that reinforcing bars may be anchored in corrugated metal ducts using an embedment depth considerably smaller than that required for development of reinforcing bars in cast-in-place concrete [1.42]. Provisions are given for bars as large as #8's, which require an embedment of 27 in. ($27d_b$) based on 5000-psi grout. Important restrictions on the use of ducts include: 1) minimum concrete side cover of 3 in., 2) minimum duct thickness of 0.023 in., 3) minimum clearance around the bar of 3/8 in., 4) grout strength not less than the concrete strength or 5000 psi, and 5) minimum embedment depth of 12 in. It is noted that alternate systems using a plastic sleeve are available. Although this would alleviate concerns over corrosion of the ducts, the author has identified proprietary plastic conduits with adequate bond characteristics that develop reinforcing bars as large as #7's.

1.6.3.2 Bars Anchored in Concrete

1.6.3.2.1 ACI 318 and Others—Straight Reinforcing Bars

For practicality, ACI 318-99 stipulates anchorage requirements for straight reinforcing bars in tension in terms of a development length requirement as follows [1.43]:

$$\frac{\mathrm{l_d}}{d_b} = \frac{3}{40}\frac{\mathrm{f_y}}{\sqrt{\mathrm{f'_c}}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)} \qquad (1\text{-}2)$$

where: $\mathrm{l_d}$=development length, in.

$\mathrm{d_b}$=nominal diameter of bar, in.

$\mathrm{f_y}$=specified yield strength of nonprestressed reinforcement, psi

$\mathrm{f'_c}$=specified compressive strength of concrete, psi

α=reinforcement location factor

β=coating factor

γ=reinforcement size factor

λ=lightweight aggregate concrete factor

This equation, based on much documented research such as References 1.45-1.47, accounts for splitting and pullout failures. The limit of 2.5 on the term $(\mathrm{c}+\mathrm{K_{tr}})/\mathrm{d_b}$ is a safeguard against pullout failure. Because a large cover can be expected for precast bent cap connections, a pullout failure, accompanied by splitting, is possible for straight bars in tension. In this case, the equation above reduces to:

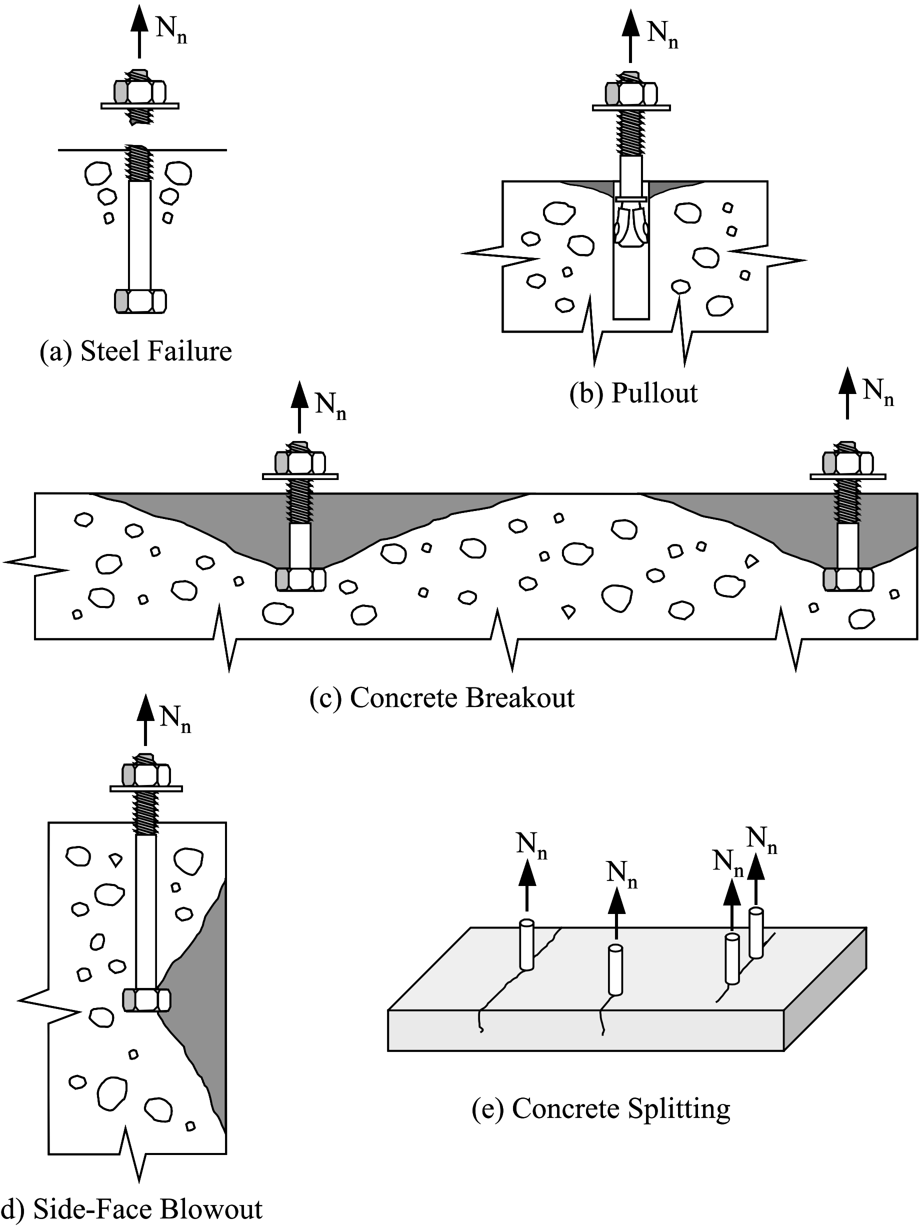
$$\frac{\mathrm{l_d}}{d_b} = 0.03\frac{\mathrm{f_y}}{\sqrt{\mathrm{f'_c}}} \qquad (1\text{-}3)$$

This is the same provision explicitly checked in ACI 318-89. This expression was based on an average bond strength of $9.5\sqrt{\mathrm{f'_c}}\,/\mathrm{d_b}$ [1.44].

1.6.3.2.2 CB-30—Concrete Breakout for Fasteners

Significant strides have been made in recent years to develop an accurate yet relatively simple and flexible approach to predict anchorage capacity of fasteners embedded in concrete. The provisions of ACI Committee 318-B (CB-30) were developed to fill such a void in the design of concrete fasteners including headed studs and bolts for which ACI 318 has previously been silent [1.40]. It is anticipated that these provisions will soon be adopted as a new appendix to ACI 318. CB-30 is based on a comprehensive review of the literature such as References 1.41 and 1.48. Fuchs et. al introduced the Concrete Capacity Design (CCD) Method [1.41], which is the basis for CB-30. A modification to this approach is used in Chapter 6 for predicting capacity of headed reinforcing bars in grout pockets. This section therefore briefly introduces the CCD Method.

The five possible failure modes for tension fasteners addressed by CB-30 are shown in Figure 1.20. Likely failure modes for headed reinforcing bars embedded in a precast bent cap system include steel failure (i.e., yield and fracture) and concrete breakout. Pullout, based on crushing at the anchor head in CB-30, and side blowout should also be checked, but are unlikely for typical conditions, especially when the contribution of bearing on lugs of reinforcing bars is accounted for. CB-30 assumes concrete splitting is prevented by required edge distances, spacing, and thickness, although the influence of splitting is investigated in the current research. Designers can specify an embedment depth and proportion variables such that the more ductile failure mode based on steel failure will govern the capacity rather than a concrete failure.



(a) Steel Failure

(b) Pullout

(c) Concrete Breakout

(d) Side-Face Blowout

(e) Concrete Splitting

Figure 1.20 Failure Modes for Fasteners under Tensile Loading [1.40]

Using a physical model based on test results, the CCD Method predicts the load-bearing capacity of a fastener or fastener group using: 1) a basic equation for tension (or shear) for a single fastener in cracked concrete, and 2) factors that account for the number of fasteners, edge distance, spacing, eccentricity, and absence of cracking. Figure 1.21 shows the assumed breakout cone for a single fastener. The basic breakout strength for a single fastener, P_b (or N_n in CB-30), is determined from a simple equation that depends only on embedment depth and the square root of the compressive strength of the concrete:

where P_b=basic concrete breakout strength in tension of a single fastener, lbs

$$= 24\sqrt{f'_c}\,h_{ef}^{1.5} \qquad h_{ef} \leq 11 \text{ in.} \qquad (1\text{-}4a)$$

$$= 16\sqrt{f'_c}\,h_{ef}^{5/3} \qquad 11 \text{ in.} \leq h_{ef} \leq 25 \text{ in.} \qquad (1\text{-}4b)$$

h_{ef}=embedment depth, in.

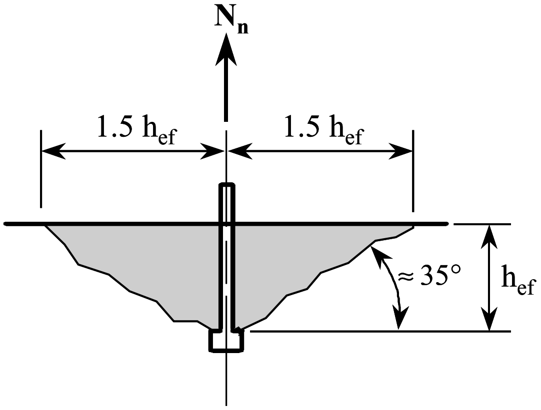


Figure 1.21 Breakout Cone for Tension [1.40]

The constants of 24 and 16 represent calibration factors based on a statistical analysis of tests and are intended to predict the 5 percent fractile (90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength). Larger coefficients are used in Chapter 6 to predict the mean breakout strength for comparison with test data. The basic equation assumes a concrete failure prism with faces inclined at an angle of approximately 35 degrees and fracture mechanics concepts (size effect) that result in the use of $h_{ef}^{1.5}$ instead of h_{ef}^{2} for shallow embedment depths. Test data indicate that a less conservative term of $h_{ef}^{5/3}$ is more accurate for deeper embedment depths.

The CCD method predicts the nominal concrete breakout strength for a group of fasteners in tension, P_{cbg}, by multiplying the basic breakout capacity by the following modification factors:

$$P_{cbg} = \frac{A_N}{A_{No}} \Psi_E \Psi_C P_b \qquad (1\text{-}5)$$

where: P_{cbg} =nominal concrete breakout strength in tension of a group of fasteners, lbs

A_N=projected concrete failure area of a fastener or group of fasteners, in.^{2}, not to exceed nA_{No}, where n is the number of tensioned fasteners in the group

A_{No}=projected concrete failure area of one fastener, when not limited by edge distance or spacing, in.^{2}

$=9h_{ef}^2$

Ψ_E=modification factor to account for edge distances smaller than $1.5h_{ef}$

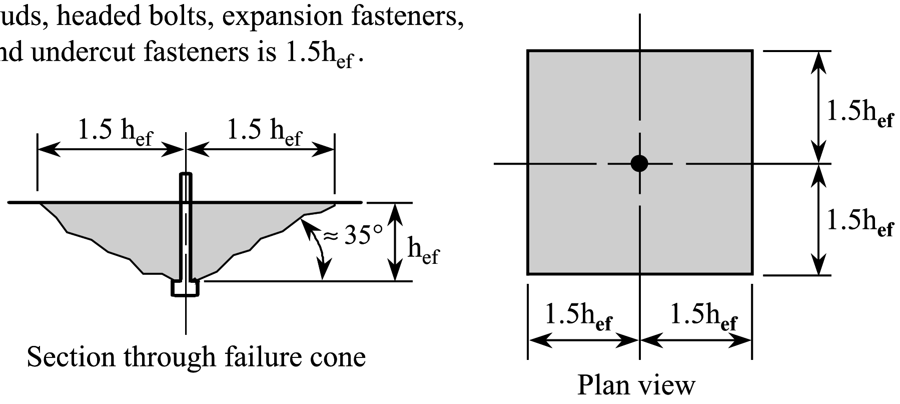
$=1$ if $c_{min} \geq 1.5h_{ef}$

$$=0.7+0.3\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < h_{ef}$$

Ψ_C=modification factor to account for cracking

c_{min}=smallest of the edge distances that are less than or equal to $1.5h_{ef}$, in.

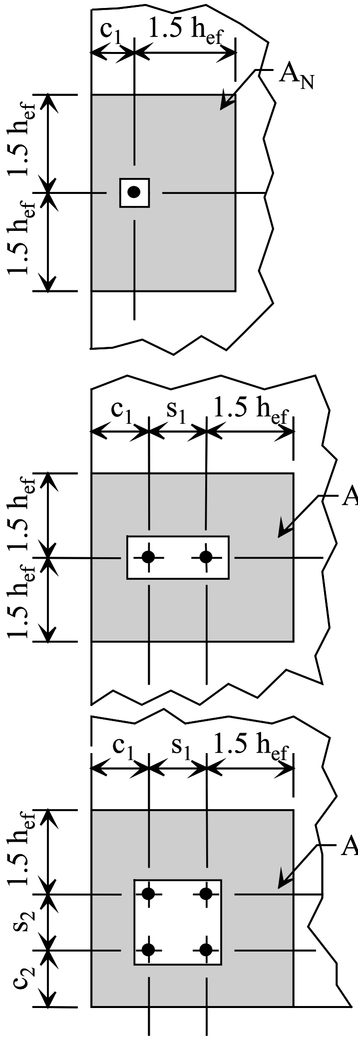
A_{No} is the maximum projected area of the failure prism for a single fastener, as shown in Figure 1.22. Because A_N is the total projected area for a group of fasteners, the term A_N/A_{No} in Equation 1-5 directly accounts for the actual failure surface, even if multiple fasteners are used and surfaces overlap. Figure 1.23 shows the calculation of the projected area, A_N, when breakout surfaces are interrupted by an edge. It has also been shown that the presence of an edge disturbs the radial stress state. The Ψ_E term reduces the capacity beyond the reduction associated with A_N/A_{No} to account for this edge effect. A Ψ_C value of 1.25 may be used for headed studs and bolts to increase the breakout capacity for cases in which cracking is not expected at service level. However, a value of 1.0 is required when cracks no wider than 0.012 in. are expected. If wider cracks are likely, confining reinforcement is required to maintain crack widths to 0.012 in. or less. Eligehausen et. al. reported a reduction in stiffness and strength of approximately 25 percent for headed studs due to the presence of cracks [1.49]. This is due to the disturbance of the stress state in the load-transfer area. Chapter 6 addresses a modification to the Ψ_C factor based on test results. CB-30 also addresses shear and interaction between shear and tension. This is further discussed in Chapter 6.



The critical edge distance for headed studs, headed bolts, expansion fasteners, and undercut fasteners is $1.5h_{ef}$.

$$\begin{aligned} A_{No} &= 2*1.5h_{ef} \times 2*1.5h_{ef} \\ &= 3h_{ef} \times 3h_{ef} \\ &= 9h_{ef}^2 \end{aligned}$$

Figure 1.22 Calculation of A_{No} [1.40]



$A_N = (c_1 + 1.5h_{ef})\ (2 \text{ x } 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})\ (2 \text{ x } 1.5h_{ef})$
if $c_1 < 1.5h_{ef}$ and $s_1 < 3h_{ef}$

$A_N = (c_1 + s_1 + 1.5h_{ef})\ (c_2 + s_2 + 1.5h_{ef})$
if c_1 and $c_2 < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

Figure 1.23 Projected Areas for Single Fasteners and Groups of Fasteners [1.40]

1.6.3.2.3 University of Texas and University of Kansas—Headed Bars

Devries and others have investigated the behavior of headed reinforcing bars in cast-in-place concrete. Devries conducted more than 140 pullout tests, mostly on single T-headed reinforcing bars developed by Headed Reinforcement Corporation (HRC) [1.50]. T-headed bars are discussed further in Section 2.3.2. Many variables were investigated such as embedment depth, edge distance, concrete strength, transverse reinforcement bar diameter, and head size and shape. The two basic failure modes other than bar yield were concrete breakout and side blowout. The CCD Method accurately predicted capacity for breakout failures. Development length of the reinforcing bars was found to increase capacity. Transverse reinforcement placed near the head increased ductility but not capacity. Tests indicated that side blowout failure would be expected for ratios of embedment depth-to-edge distance greater than approximately 5 for a 2 in. x 2 in. head. CB-30 requires a check of side blowout for a ratio of 2.5. The performance of bars with a smaller head (upset-headed bars), also produced by HRC, has not been published in the available literature. These bars are also discussed in Section 2.3.2.

Wright et. al. conducted 70 beam-end tests on HRC T-headed reinforcing bars [1.51]. Comparisons were made with straight and hooked bars. When bars were not debonded, behavior and capacity of headed bars were found to be similar to that of hooked bars. Headed bars with two bar diameters of cover were developed in a length approximately 60 percent of that required for a hooked bar. Confinement provided by cover or ties around the head was found to significantly enhance anchorage. In contrast to conclusions of Reference 1.50, debonding of bars was found to increase anchorage capacity by preventing the formation of splitting cracks due to bearing along the lugs.

1.6.3.2.4 University of Texas –Cyclic Loading

As part of a study of headed bars in joints, Bashandy conducted fourteen pullout tests to investigate the effects of tension cyclic loading and anchoring the head behind a crossing bar on anchorage behavior [1.52]. All bars were sheathed to prevent bond along the bar length. Cycling the load 15 times in a range between 5 percent and 80 percent of the capacity reduced the anchorage capacity of headed bars less than 10 percent. Increase in head slip depended on the maximum load for each cycle. For bars cycled to a load less than the elastic range (based on load-slip behavior), the increase in slip was minimal. For cycling at larger loads, up to a 35 percent increase in slip occurred. Cook [1.53] found that high-cycle fatigue loading has no effect on anchor strength when anchors (both cast-in-place and grouted threaded rods) are embedded sufficiently to develop full tensile capacity of the anchor steel under static loads. CB-30 excludes load applications that are predominantly high cycle fatigue, but not seismic load effects.

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CHAPTER 2:
DEVELOPMENT OF CONNECTION DETAILS

2.1 INTRODUCTION

Based on a review of relevant literature and existing bridges using precast bent caps, several candidate precast connection details were developed. As mentioned in Chapter 1, details must be economical and satisfy requirements of constructability, durability, and force transfer. To help ensure details achieve this standard, University of Texas researchers developed connection details in coordination with an advisory board that included representatives of the precast and construction industries as well as TxDOT engineers. Table 2.1 lists the six members of the Industry Review Committee (IRC) and three TxDOT engineers who reviewed preliminary details and helped refine them through two review cycles. The connection details documented in this chapter are considered the most promising details for a precast bent cap system, several of which are investigated in Phase 1-3 tests.

Table 2.1 Industry Review Committee and TxDOT Engineers

Name	Organization
Randy Rogers, Chairman	McCarthy Brothers, Austin
Charlie Burnett	Champagne-Webber, Houston
Paul Guthrie	Texas Concrete, Victoria
Fred Heldenfels, IV	Heldenfels Enterprises, San Marcos
Carl Thompson	Dalworth Concrete, Houston
Roger Welsh	Assoc. of General Contractors, Austin
John Vogel, Project Director	Texas Dept. of Transportation, Houston
Robert Sarcinella, Project Advisor	Texas Dept. of Transportation, Austin
Lloyd Wolf, Project Advisor	Texas Dept. of Transportation, Austin

Connection details fall into one of four general connection categories: 1) grout pockets, 2) grouted vertical ducts, 3) grouted sleeve couplers, and 4) bolted connections. Successfully developed details may be considered a family of connection alternatives for a precast bent cap system. The particular connection type and selected detail would depend on the specific project application. As mentioned in Chapter 1, a precast bent cap system in Texas is expected to use either a rectangular or inverted-tee cap on cast-in-place columns or precast trestle piles. For a typical trestle pile bent with a rectangular cap, a detail from any connection category can provide adequate transfer of forces. Consideration of economics and constructability may lead, for instance, to the use of a single-line or double-line grout pocket. In contrast, for an inverted-tee cap on cast-in-place columns, grout pockets would probably be eliminated because of conflict between cap reinforcement and the necessary voids. In this case, the designer might choose a grouted vertical duct or bolted connection.

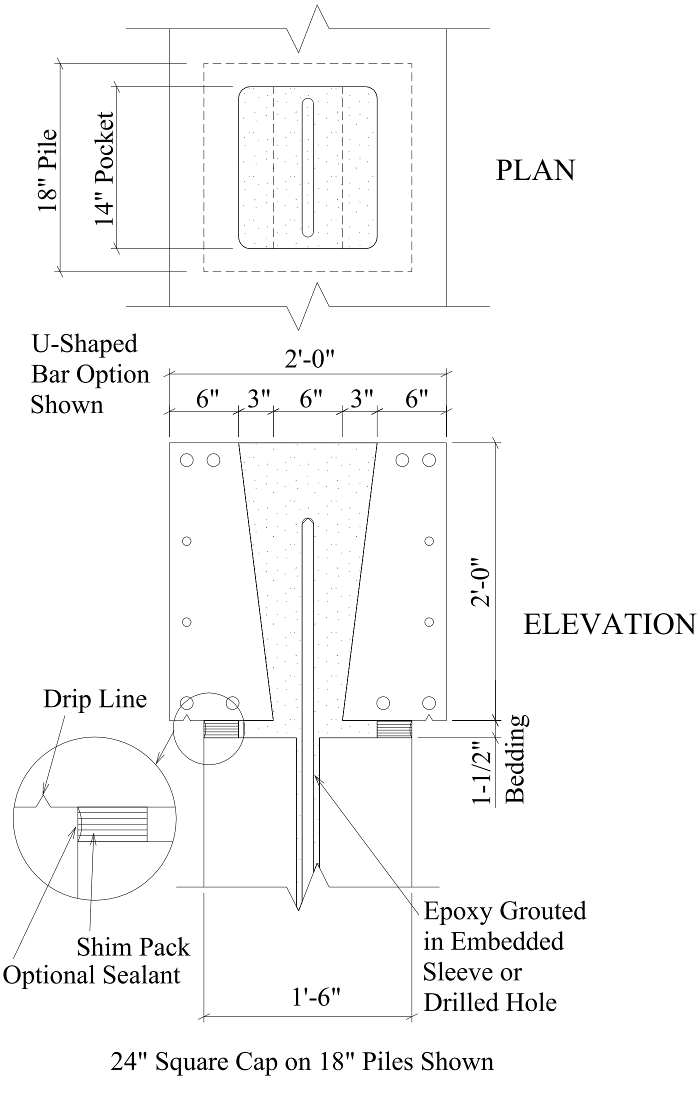
The main components defining a precast bent cap connection are the connection type, connectors, and grout used to complete the connection. The following sections of this chapter describe the development of these three components for precast bent cap systems. This is the basis for the connection details used in testing.

2.2 CONNECTION CATEGORIES

This section introduces the features of select details from each of the four connection categories and provides a summary of the primary advantages of each detail. A synopsis of major topics discussed during IRC review cycles is then presented. Finally, disadvantages and uncertainties associated with each connection type are discussed. Further background development of these and other connection details may be found in Reference 2.1.

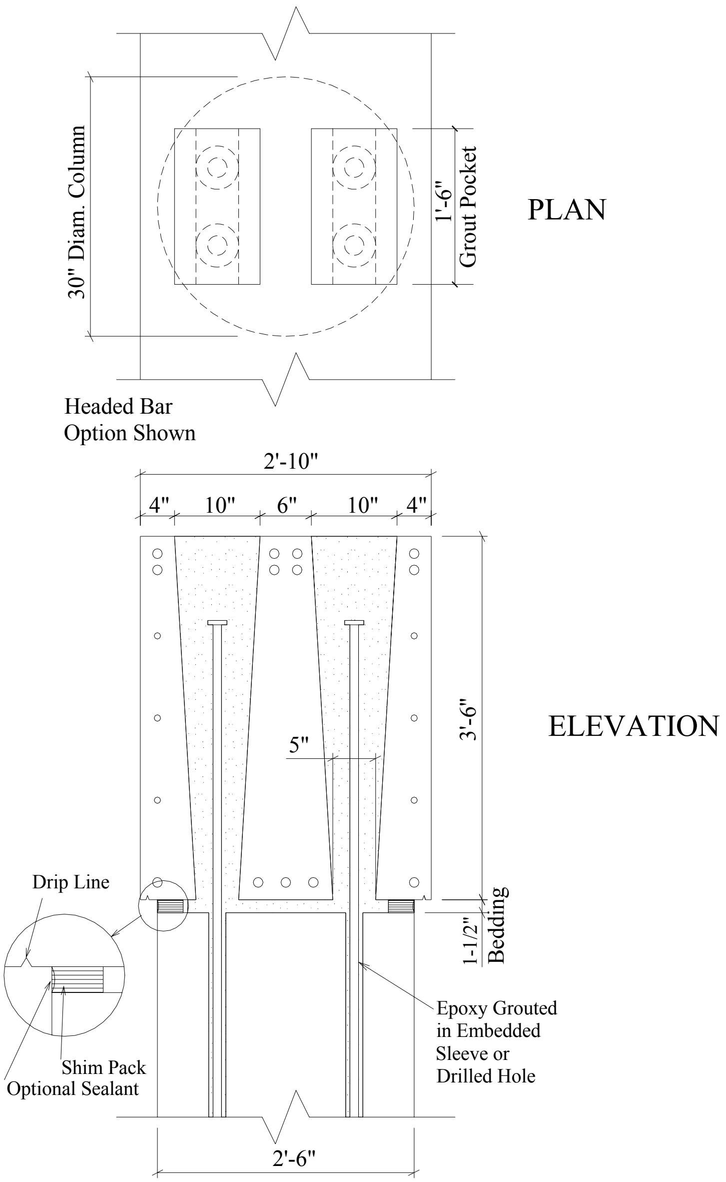
2.2.1 Grout Pockets

Figures 2.1 and 2.2 show plan and cross-section views of typical single-line and double-line grout pocket connections, respectively. Grout pocket connections derive their name from the fact that they incorporate precast voids, or pockets, in the bent cap to accommodate, i.e. house, connectors extending out of top of columns or piles. After placement of the cap over the connectors, the pocket is grouted.



24" Square Cap on 18" Piles Shown

Figure 2.1 Single-line Grout Pocket Connection



34" x 42" Cap on 30" Diam. CIP Column Shown

Figure 2.2 Double-line Grout Pocket Connection

2.2.1.1 Advantages

The use of grout pocket connections has many advantages:

- accommodation of large tolerances to provide "forgiveness" of construction errors
- pocket shapes that can be tailored for specific project requirements including constructability, durability, and force transfer
- use of materials with well-understood behavior
- successful previous use in building construction and, to a limited extent, in bridge construction

The ability to design "forgiveness" of construction error into connections and to tailor pockets for specific applications is a major advantage.

2.2.1.2 Features

The single-line and double-line grout pocket connections shown in Figures 2.1 and 2.2 are examples of grout pocket connections. The precast cap with a single-line grout pocket is shown as part of a trestle pile bent, whereas the cap with double-line grout pockets is set on cast-in-place columns. It is expected that typical grout pocket connections for precast caps on cast-in-place columns will require two rows of connectors to provide adequate transfer of forces, while the designer may have the option of using either a single-line or double-line of connectors to accommodate the lower force transfer requirements for trestle pile bents.

The single-line grout pocket connection uses a truncated pyramid-shaped grout pocket that has a single taper in both directions. This may be formed with removable and reusable steel forms. Although steel produces a smooth surface at the interface, the seven-degree taper through the cross section of the cap enhances interlocking of the grout pocket, and thus the connectors, within the cap when the connectors are subjected to tension. The taper in the perpendicular direction is approximately two degrees, enough for form removal. The pocket is sized to provide horizontal tolerances of about 2.5 in. in the longitudinal direction and 1 in. in the transverse direction. A 1.5-in. bedding layer between the cap and pile accommodates vertical tolerances and grouting of the bedding layer. Figure 2.1 shows the connector as a U-shaped bar, although a variety of connector options exist, including as headed, hooked, and straight bars. Connectors are described in Section 2.3. A non-shrink cementitious grout is used to complete the connection.

The construction sequence (shim option) is as follows:

1. drive piles with template and break back to desired elevation
2. grout anchor bars into embedded sleeves or drilled holes
3. set shims and verify elevation
4. lower cap over connectors onto shims
5. form around bedding layer
6. grout bedding and pockets through opening at top of cap

This construction sequence is the same as that used in Red Fish Bay, except that shims rather than friction collars are shown to support the cap.

As shown in Figure 2.2, the double-line grout pocket connection uses two, side-by-side pockets. To minimize interference with the cap longitudinal reinforcement, the taper angle is approximately half of that used for the single-line pocket. Tolerances and the bedding layer thickness are similar to those used for the single-line pocket. The figure shows headed bars used as connectors. The construction sequence would be essentially the same as that for the single-line connection.

2.2.1.3 IRC Review Topics

A large number of refinements were made to initial connection details to arrive at the grout pocket connections shown in Figures 2.1 and 2.2. Refinements were based on discussions during the IRC review cycles. The major topics are briefly discussed in this section, as they relate to constructability, durability, and force transfer for grout pockets. Some of these issues apply to other connection types as well. Review of these comments helps to: 1) illustrate considerations of the team during review cycles, 2) provide rationale leading to the decisions for the details selected for testing, and 3) explain the requirement for certain instrumentation used in testing.

2.2.1.3.1 Constructability

Figure 2.3 shows three additional grout pocket shapes that were considered: a double bevel, a funnel-and-tube, and a truncated pyramid-shaped pocket that is not inverted. The double bevel pocket, similar to that used in Red Fish Bay, was developed to possibly enhance anchorage of connectors and to simplify form removal. The funnel-and-tube pocket was considered to provide reliable anchorage, but would require more costly stay-in-place forms. The larger base of the pyramid-shaped pocket was intended to better accommodate potential grout shrinkage than the inverted pyramid-shaped pocket, but could significantly reduce interlock between the pocket and cap.

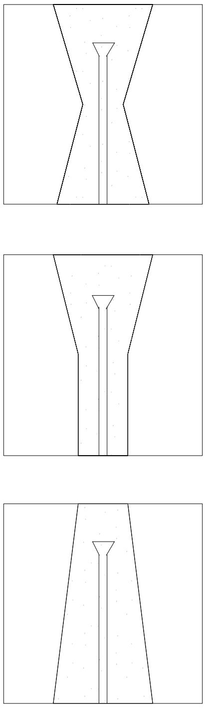


Figure 2.3 Additional Grout Pocket Configurations

Interference with typical cap reinforcement was also a concern. Single-line pockets with large taper angles through the cross section may lead to congestion of longitudinal bars. A viable alternative is to provide a large pocket taper along the length of the cap, rather than through the cross section. As shown for the double-line grout pocket, even the use of a relatively small taper angle can block a significant amount of the cross section. Grout pockets may also interfere with inverted-tee cap ledge reinforcement, even for shallow pocket heights.

Because of the uncertainty of adequate interlock between the grout pocket and cap, roughened pocket surfaces were considered. Roughened surfaces have been used in precast construction by spraying a paste-dissolving compound on the formwork and air blasting the surface after precasting, thereby exposing coarse aggregate. Roughened interfaces are certain to enhance the pocket-to-cap interlock, but increase the cost of fabrication. Smooth interfaces would provide a lower, yet perhaps acceptable, level of interlock, and are easily produced with removable forms.

Placement of the cap over piles versus on top of piles (or columns) was also discussed. In the Red Fish Bay project, the grout pocket was shaped and sized so that the cap could be lowered over and enclose the top of the pile. This detail requires more extensive field operations, such as the use of a template for pile driving to ensure accurate alignment of piles. Visual inspection of the adequacy of grouting operations at the bedding layer is not possible for this connection. In addition, cracking and/or deterioration of the connection cannot be easily monitored or repaired. On the other hand, this detail may be more durable, because a more limited path for moisture ingress exists. When a cap is set above piles or columns, the bedding layer may be inspected, but is exposed to the environment.

Two basic grouting approaches were considered: 1) gravity-flow grouting from an opening at the top or side of the cap, and 2) pressure grouting from the bedding layer upwards. For rectangular caps, this would involve grouting from the cap top. For inverted-tee caps, grouting through sidewall grout ports may be preferable for pockets that are less than full height, due to the large cap depth. Gravity-flow grouting is a decidedly simpler approach to fill the pocket, but requires care during grouting operations to ensure the bedding layer and pocket are grouted without air voids. Pressure grouting, on the other hand, requires a higher level of skilled labor in the field and is more costly, but would more likely result in a pocket free of voids. For either approach, venting of air at the bedding layer would be required. Grouting operations and grout performance requirements are discussed further in Section 2.4 and subsequent chapters.

Use of shims versus friction collars to support the precast cap was also considered. Shims are inexpensive and readily available, and are fairly simple to place. However, due to insufficient bearing area on the cap, shims were found to be unstable during cap setting operations for Red Fish Bay, and friction collars similar to that shown in Figure 2.4 were later used. With the larger bearing area at the underside of cap for Pierce Street Elevated bents, shims were found to be adequate (Figure 2.5). Additional uncertainty exists for shims because of the unknown impact of shim "hard spots" at the column-cap interface. If connection grout tends to shrink away from the cap, excessive bearing stresses may develop at the shim. Friction collars eliminate this concern. Friction collars are already commonplace for cast-in-place bents, and typically deliver a capacity up to 100 kips/column. Capacities as large as 280 kips/column are available for bolt-on brackets. Collars have the advantage of easy adjustment and provide temporary support while connection grout cures, but are a more costly option. With shims, shoring may be required for cap stability while the connection grout cures.

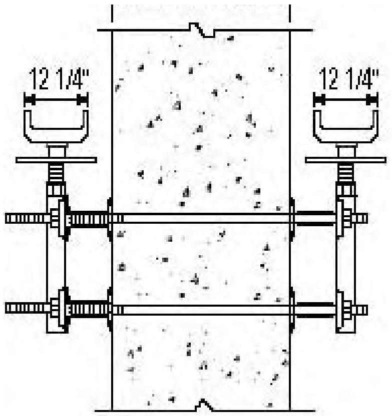


Figure 2.4 Friction Collar

Figure 2.5 Shims at top of Column in Pierce Street Elevated Project

2.2.1.3.1 Durability

Durability issues are primarily related to corrosion of the connectors and other connection hardware due to: 1) voids, gap opening, and cracks at the bedding layer, 2) cracks at the top surface of the grout pocket, and 3) inadequate grout properties. Figure 2.6 shows these issues schematically. Most of the IRC comments regarding durability apply to all connections.

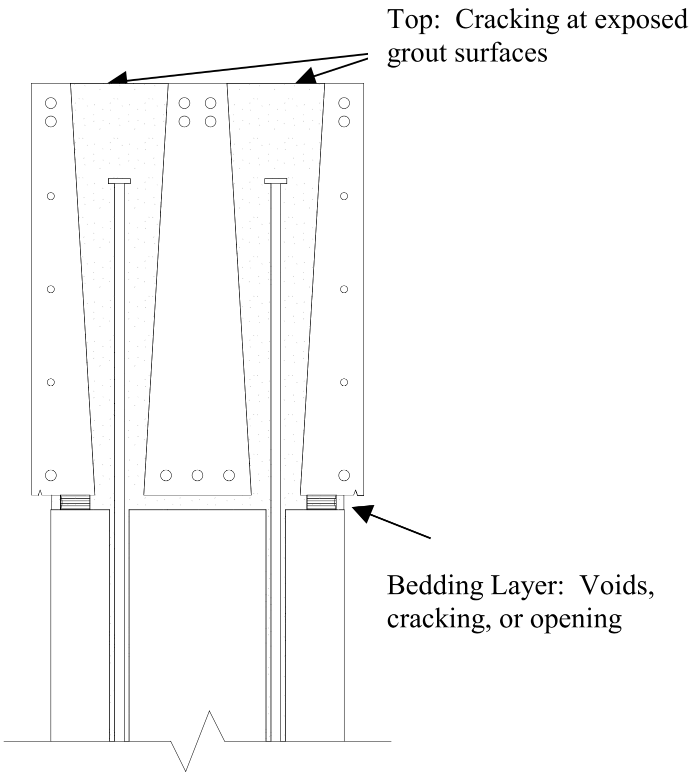


Figure 2.6 Durability Issues for Precast Bent Cap Systems

It was agreed that testing will help determine the extent of bedding layer opening, cracks, and voids, as well as cracking at the top surface of the grout pocket under service loads. If necessary, an external sealant may be applied to waterproof exposed surfaces. Other provisions such as a waterstop joint were thought to add undue complexity and cost. Recent inspection of cracking at the top of grout pockets at Red Fish Bay revealed very sparse, small cracks with a maximum width of only 0.005 in. [2.2].

Use of plain, uncoated steel reinforcement in a connection may corrode. In many applications, connectors and connection hardware may be epoxy-coated or galvanized. Use of epoxy-coated bars follows TxDOT's "belt and suspenders" approach to durability and provides one of several possible barriers to corrosion. Steel shims also may contribute to corrosion, and thus plastic shims should be investigated. Plastic shims have the added benefit of preventing a potential "hard spot" in the connection, since plastic shims will creep and are expected to better transfer connection loads to the grout.

All moisture paths into a connection should be prevented. This means air voids should be eliminated by proper grout mix specification and grouting operations such as air venting. The grout mix should also be designed for durability, accounting for shrinkage, permeability, and flowability. For some applications, grout may also provide freeze thaw and sulfate resistance. Prepackaged grouts that are carefully selected are economical. In addition, drip beads that prevent water from washing over the bedding layer may be included into the cap design.

2.2.1.3.2 Force Transfer

Discussions of force transfer centered on several important issues, some of which apply to other connection types as well. Anchorage of bars within the grout pocket and interlock of the grout pocket within the precast cap are critical connection issues. Testing should help determine the adequacy of several grout pocket shapes and heights, as well as requirements for embedment and surface roughening. Joint confining reinforcement may contribute to achieving ductile connection behavior. Reinforcement provisions are not obvious and should be examined through testing.

Redundancy for precast connections is important and advantageous. Single-line grout pockets allow more connectors if straight or headed bars are used, rather than U-shaped bars. In addition, to increase redundancy, a designer may use a double-line grout pocket, even if calculations show a single-line pocket works. Alternatively, designers may apply larger strength reduction factors.

2.2.1.4 Disadvantages and Uncertainties

Disadvantages and uncertainties exist for grout pocket connections. Disadvantages include:

- large exposed surface areas at the cap top, posing potential durability concerns
- relatively large grout volume requirements
- increased congestion of longitudinal reinforcement, due to pocket interference in cross section
- applications likely limited to shallow bent caps

Based on committee comments, it is evident that there are a number of uncertainties associated with grout pocket connections, due primarily to the limited experience related to construction and behavior of these connections. Major uncertainties include:

- anchorage of bars within the grout pocket
- interlock of the grout pocket within the precast cap
- performance compared with monolithic construction
- failure modes

- influence of confinement reinforcement
- gravity-flow grouting of the bedding layer and pockets
- grout performance requirements
- influence of bedding layer shims on connection behavior
- cracking at the surface top
- cracking, gap opening and voids at the bedding layer

Phase 1-3 tests are intended to address these uncertainties, as discussed in Chapters 3-5.

2.2.2 Grouted Vertical Ducts

As the name implies, grouted vertical ducts are connections that use corrugated ducts placed vertically through the bent cap cross section to house column connectors. These connections are grouted after placement of the cap. Figure 2.7 shows plan and cross section views of a grouted vertical duct connection in an inverted-tee cap.

2.2.2.1 Advantages

Because grouted vertical ducts are similar to grout pocket connections, many of the grout pocket advantages apply also to these connections. Advantages include:

- accommodation of large tolerances to provide "forgiveness" of construction errors
- inexpensive, stay-in-place steel corrugated ducts that may be specified to satisfy constructability and force transfer requirements
- minimal duct interference with cap reinforcement
- use of materials with fairly well understood behavior

2.2.2.2 Features

The connection shown in Figure 2.7 is an example of a possible grouted vertical duct connection for a precast bent cap system. This connection may be used for rectangular or inverted-tee caps, on either trestle piles or cast-in-place columns.

In contrast to grout pocket connections, which use removable steel forms to form a single-taper pocket, grouted vertical duct connections use stay-in-place, steel corrugated ducts to house connectors. This facilitates cap fabrication. Common to the post-tensioning industry, steel corrugated ducts are inexpensive and readily available. Although minor variations in wall thickness, corrugation pattern, and duct diameter exist for different manufacturers, suitable bond and strength are expected. With standard diameters as large as approximately 5 in., ducts provide ample construction tolerances for even the largest column bars. A 4-in. duct such as that shown in Figure 2.8 would provide approximately a 1-in. horizontal tolerance with headed #11 bars. Additional benefits of the grouted vertical duct connection include the vertical orientation of the duct, which limits interference with cap reinforcement, and the use of steel ducts, which may enhance connector confinement.

For the large depths of inverted-tee caps, ducts may be either partial-depth or full-depth (compare Figures 2.7 and 2.9). Partial-depth ducts may be gravity-flow grouted through grout tubes in the cap sidewall or pressure grouted from the bedding layer. Rectangular caps are likely to use full-depth ducts due to the relatively shallow cap depths. While this facilitates the use of gravity-flow grouting, it leaves the top surfaces of the ducts exposed.

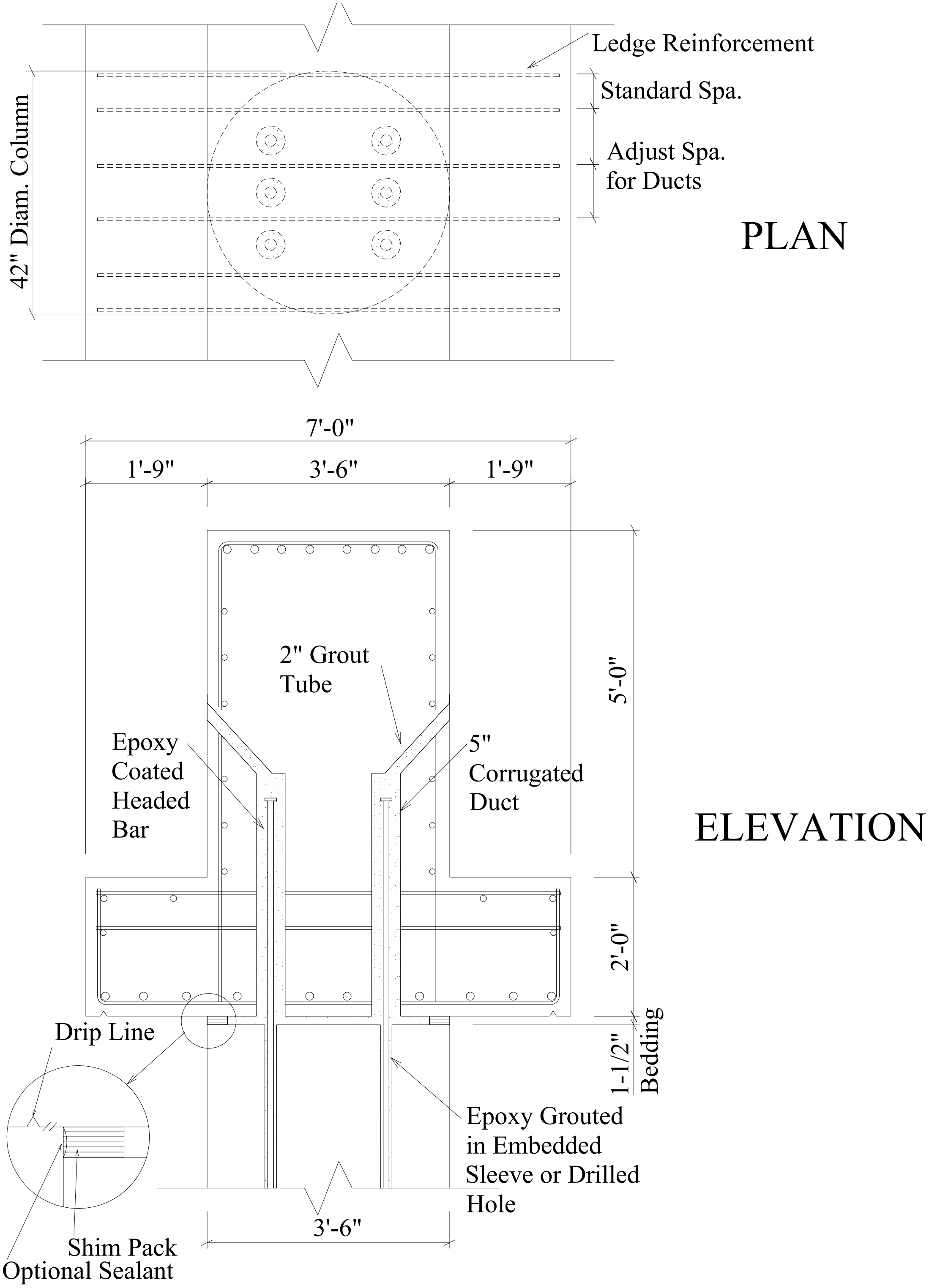


Figure 2.7 Grouted Vertical Duct Connection (Inverted-Tee Cap)

Figure 2.8 4-in. Diameter Steel Corrugated Duct

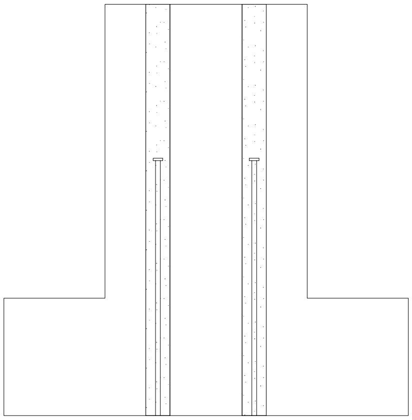


Figure 2.9 Full-Depth Duct in Inverted-Tee Cap

The construction sequence for connections grouted from the cap top (shim option) is as follows:

1. cast columns in place with embedded sleeves or drill holes
2. grout connectors into embedded sleeves or drilled holes
3. set shims and verify shim elevation
4. lower cap over connectors onto shims
5. form around bedding layer
6. grout bedding layer and ducts through opening at cap top or through grout ports

An alternative construction sequence for this connection is possible. Instead of embedding sleeves in the cap, the cap could serve as a template for drilling holes into the column. After cleaning out the holes and ducts, connectors would be grouted into holes, and steps similar to the above construction sequence would be followed. Although this may increase the number of construction operations, it eliminates tolerance issues associated with connectors. Either construction sequence could use friction collars or shoring instead of shims.

2.2.2.3 IRC Review Topics

Development of the grouted vertical duct connection was an outcome of the grout pocket discussions. Because of this, there were not separate IRC comments for grouted vertical ducts; rather, many of the topics mentioned previously apply to this connection.

2.2.2.4 Disadvantages and Uncertainties

There are few obvious disadvantages for grouted vertical ducts. One disadvantage is the exposed surface areas at the cap top, when grouting is performed from the top. However, this area is relatively small. If serious durability concerns exist, a waterproofing agent may be applied.

Most concerns center on uncertainties that are addressed in Phase 1-3 testing. Major uncertainties include:

- anchorage of bars within ducts
- anchorage of ducts within the cap
- performance compared with monolithic construction
- failure modes
- influence of duct and cap confining reinforcement
- gravity-flow grouting of multiple ducts through the bedding layer
- grout performance requirements
- influence of bedding layer shims on connection behavior
- cracking at the top surface of the duct
- cracking, gap opening and voids at the bedding layer

2.2.3 Grouted Sleeve Couplers

Grouted sleeve couplers, such as the NMB splice sleeve system and Erico's Lenton *Interlok* mechanical reinforcing steel splice, are proprietary metal couplers that have been used for years in precast building construction. Cylindrical sleeves are cast into the precast element and are grouted after the connecting reinforcement is inserted into the sleeves during erection. Figures 2.10 through 2.12 [2.3,2.4] show grouted sleeve coupler details and applications.

2.2.3.1 Advantages

Advantages of grouted sleeve couplers include:

- successful previous use in building construction
- reliable development of anchorage and full continuity of column bars, established by testing
- ductile mode of failure
- limited exposed surfaces

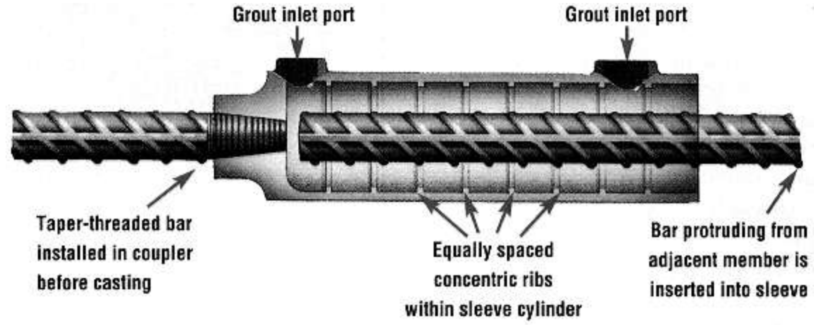


Figure 2.10 Lenton *Interlok* Grouted Sleeve Coupler [2.3]

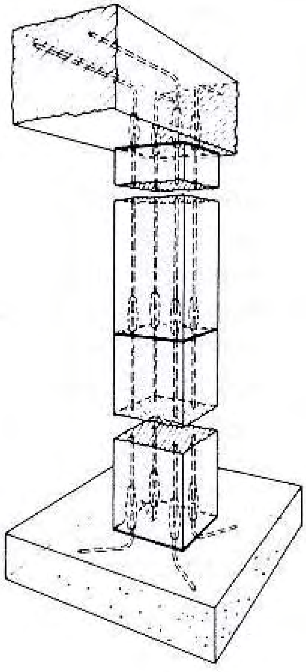


Figure 2.11 NMB Grouted Sleeve Coupler used on Precast Bridge Bent [2.4]

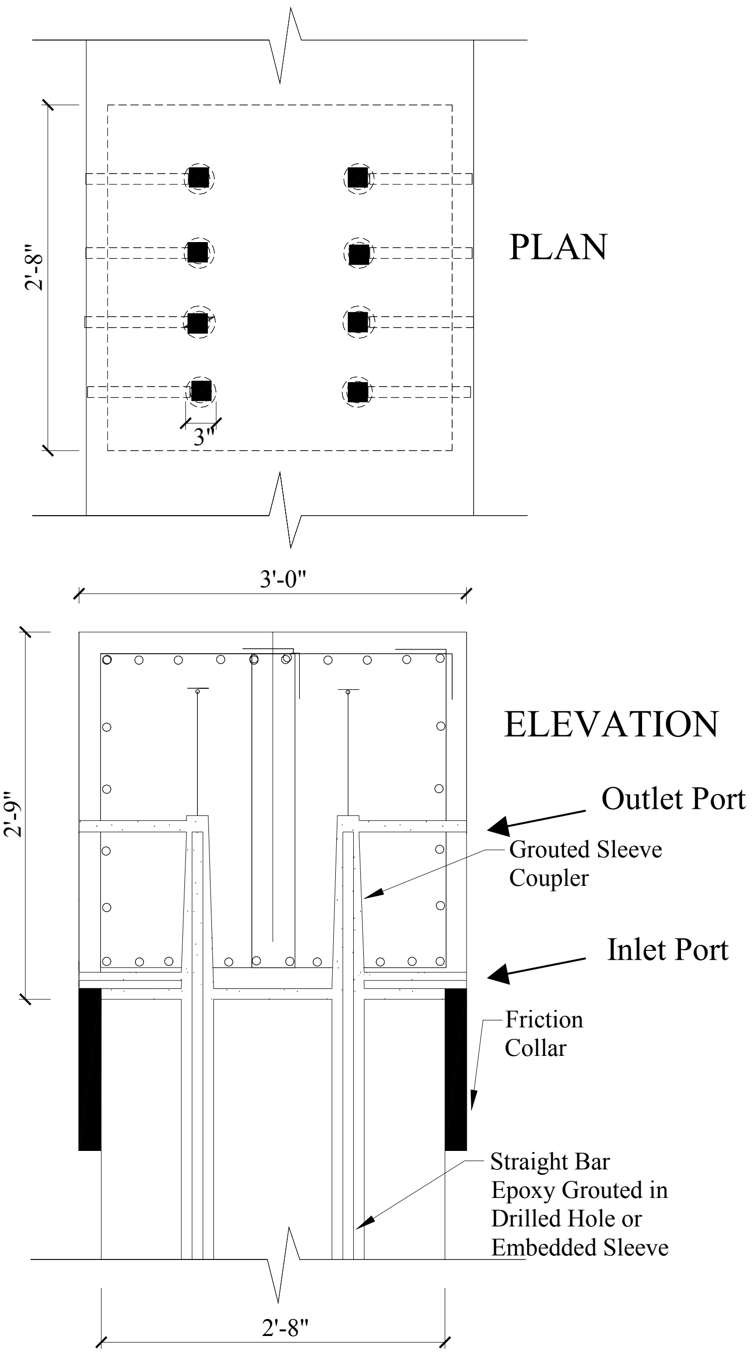


Figure 2.12 Grouted Sleeve Coupler Connection

2.2.3.2 Features

The grouted sleeve couplers shown in the figures are examples of couplers and connection details that could be implemented in a precast bent cap system. Grouted sleeve couplers appear to provide a simple and reliable solution for precast connections; however, for bridge applications, they require satisfying extraordinarily tight horizontal tolerances no larger than approximately ¾ in. and often considerably smaller. After placement of the cap, two grouting operations are required, one for the bedding layer and another for the sleeves.

The construction sequence for grouted sleeve couplers (shim option) is as follows:

1. cast columns in place with embedded sleeves or drill holes
2. grout connectors into embedded sleeves or drilled holes
3. set shims and grommet washers and verify shim elevation
4. lower cap over connectors onto shims
5. form around bedding layer
6. grout bedding layer
7. pressure grout sleeves

An alternative construction sequence is to support grommet washers against the base of the cap using shims or wedges and then grout the sleeves [2.4]. After the grout sets, grommet washer supports are removed and the bedding is grouted. Both approaches are feasible.

A preferable construction sequence to either of the above mentioned alternatives would be to grout the bedding and couplers with the same grout, thereby simplifying field operations. However, to produce high strength, non-shrink grout, manufacturers add metal aggregate that cannot be exposed to the environment. Therefore, different grouts must be used in separate grouting operations.

2.2.3.3 IRC Review Topics

Review comments revealed several issues that could significantly increase construction cost. The tight horizontal construction tolerances typically less than ¾ in. were considered a major drawback. Accommodating such tolerances could be very challenging, especially with driven piles. It should be noted that these tolerances have been achieved in previous projects in other states. As mentioned in Chapter 1, contractors involved in the Edison Bridge project in Fort Meyers, Florida used templates with NMB grouted sleeve couplers to successfully connect H-shaped precast columns to inverted U-shaped bent caps. However, this project required special provisions such as the use of a template to align column bars between precast components, exceptional precaution by contractors, and additional coordination between fabricators and contractors.

Other factors could also increase cost. Grouted sleeve couplers are relatively expensive items. In addition, because the couplers and grout are proprietary products, contractor options would be limited to the few available products. Grouted sleeve couplers manufactured by NMB and Erico are the only couplers known to have been used in precast bridge projects in the U.S. Also, grout pumping would require extra equipment and greater quality control. The requirement for separate grouting operations for the sleeve and bedding layer would also increase cost.

Two issues associated with force transfer are of interest. Performance equal to monolithic construction is expected when grouted sleeve couplers are used. NMB has stated that behavior of structures using the NMB grouted sleeve coupler can emulate monolithic cast-in-place structures when high-quality bedding materials are used [2.5]. Another issue is the anchorage of sleeves within the precast cap, which may be accomplished with hooked or headed bars. Because trestle pile caps typically use shallow depths, anchorage requirements for these bars would require verification.

2.2.3.4 Disadvantages and Uncertainties

Disadvantages for grouted sleeve couplers center on construction challenges that increase cost:

- excessively tight horizontal tolerances, especially for trestle pile bents
- proprietary hardware and grout with limited alternatives
- limited experience in bridge construction

- higher construction skill
- required grout pumping
- multiple grouting operations for the sleeves and bedding layer

Few uncertainties exist for grouted sleeve couplers because the system is tested and proven. Uncertainties include:

- bedding grout performance requirements
- influence of bedding layer shims on connection behavior
- cracking, gap opening and voids at the bedding layer

2.2.4 Bolted Connections

Bolted connections for precast bent caps introduce several aspects not usually associated with bolted connections, particulary the construction sequence and grouting. Figures 2.13 and 2.14 show plan and cross section views of bolted connections that can be implemented in a precast bent cap system, including a shim and leveling nut option.

2.2.4.1 Advantages

Bolted connections possess many advantages. Because of the obvious similarities to grouted vertical ducts, there are several common advantages, such as:

- accommodation of large tolerances to provide "forgiveness" of construction error
- familiar construction practices
- inexpensive, stay-in-place steel corrugated ducts that can be specified to satisfy constructability and force transfer requirements
- minimal duct interference with existing cap reinforcement
- use of materials with well understood behavior

Additional advantages unique to bolted connections include:

- successful previous use in building construction
- resistance of large connection moments by use of high strength bolts
- load path redundancy through bond and cap top anchorage
- temporary support during erection
- full continuity of column bars through the connection
- optional post-tensioning to enhance durability and proof load installation

Provision for large moment resistance is especially useful for inverted-tee caps. Load path redundancy and erection support are also key advantages.

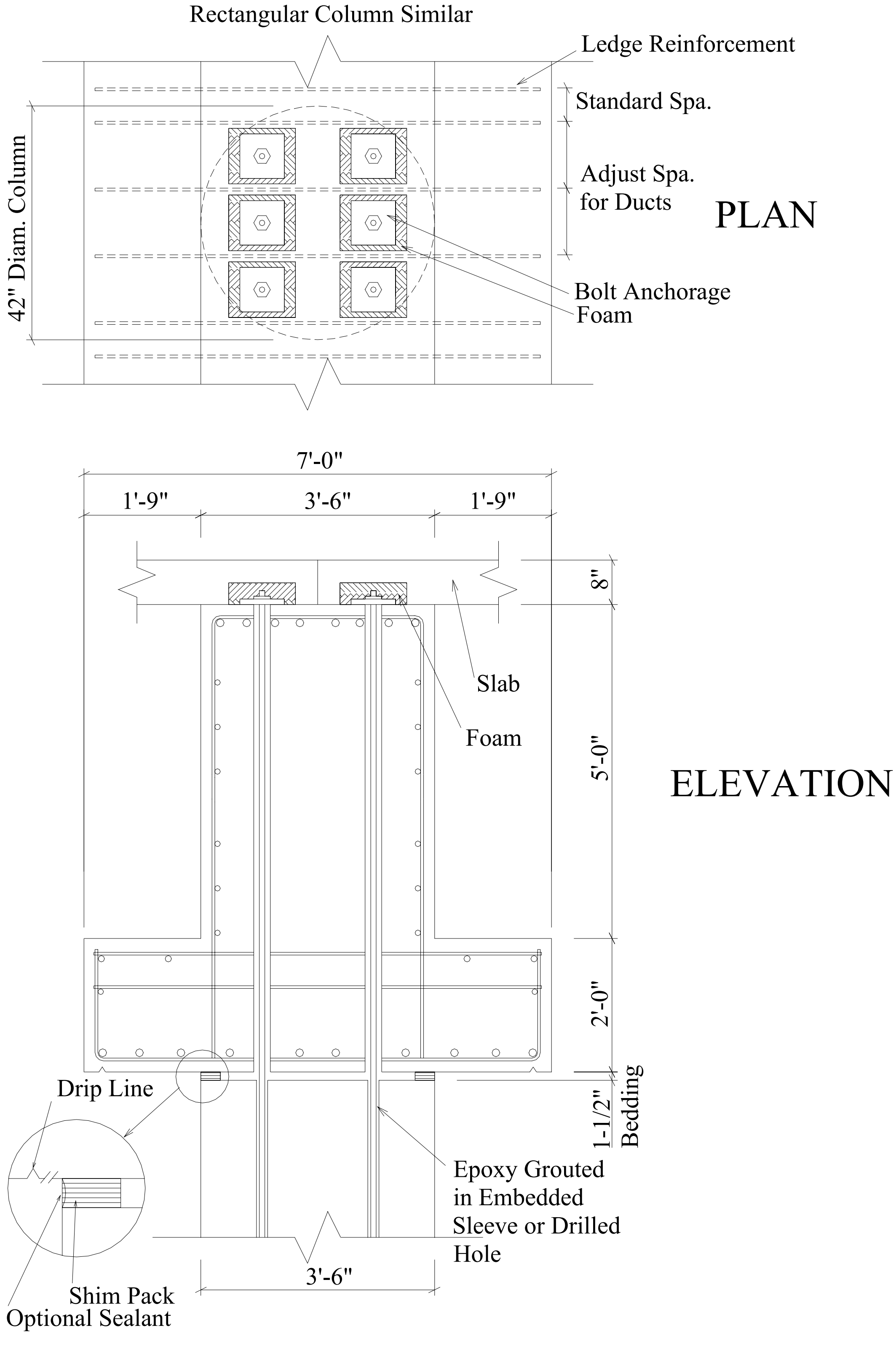


Figure 2.13 Bolted Connection (shim option)

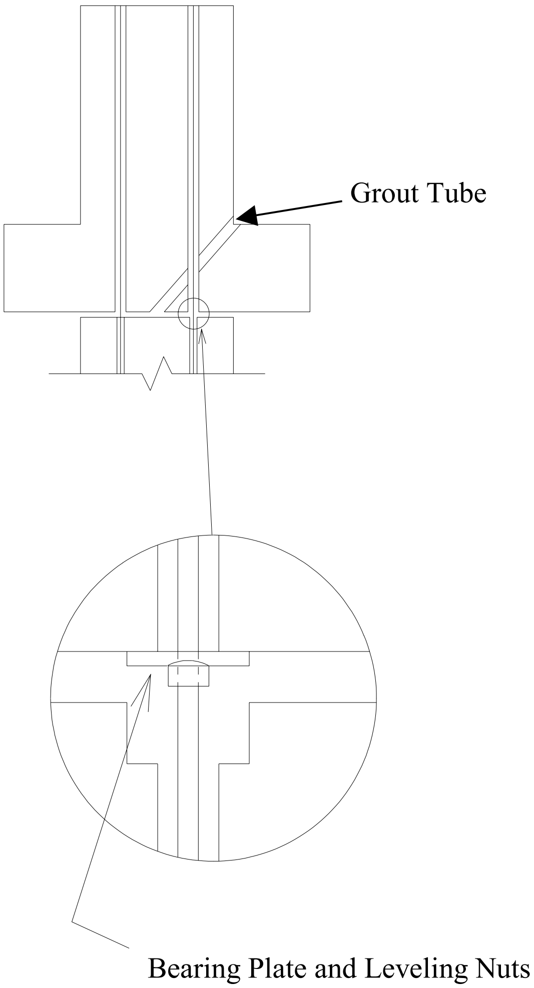


Figure 2.14 Bolted Connection (leveling nut option)

2.2.4.2 Features

In Figures 2.13 and 2.14, the bolted connection alternatives are applied to inverted-tee caps. These connections may also be used with rectangular caps on either cast-in-place columns or precast piles.

Bolted connections use stay-in-place, steel corrugated ducts to accommodate connectors that extend through the depth of the cross section and are anchored at plates at the top of the cap. In addition to providing temporary support during erection, this approach has the advantage of load path redundancy, because both bond and anchor plate bearing contribute to anchorage.

Bolted connections on the Pierce Street Elevated project used 3-in. ducts, but larger diameter ducts that provide greater construction tolerances may be used. Durability concerns over the steel anchorage at the cap top may be addressed by protecting the anchorage within an encasement or by using protective coatings. An encasement such as that shown in Figure 2.13 would also accommodate relative movement between the slab and anchorage due to thermal expansion.

A notable difference between the two details displayed in the figures is the approach to support the cap during erection. Figure 2.13 shows the use of shims, whereas leveling nuts with bearing plates support the cap in Figure 2.14. Shims, friction collars, or leveling nuts with plates are all viable alternatives. Threaded bolts aid the use of leveling nuts, which can be "dialed in" to the required elevations for precision placement of the cap. The bedding can be reliably grouted through an embedded grout tube that extends from the sidewall to the center of the bedding layer, similar to that successfully used in the Pierce Street Elevated bent caps. Unless bearing plates are designed to permit flow through the ducts, the bedding layer and ducts would have to be grouted separately.

The construction sequence for the bolted connection with shims on a cast-in-place column is as follows:

1. cast columns in place with embedded sleeves or drill holes
2. grout connectors into embedded sleeves or drilled holes
3. set shims and verify shim elevation
4. thread cap over bolts onto shims
5. form around bedding layer
6. grout bedding and ducts through duct opening at cap top
7. tighten bolts

Threading of the cap over the bolts requires special procedures to ensure all bolts pass successfully through the cap. Also, grouting of the ducts and bedding may be completed through a single duct, allowing other bolts to remain tightened for stability during erection. In this case, bearing plates require air venting. In addition, each bolt may have to be loosened and later re-tightened to allow ducts to be topped off with grout.

When leveling nuts with bearing plates are used, the construction sequence changes:

1. cast columns in place with embedded sleeves or drill holes
2. grout connectors into embedded sleeves or drilled holes
3. adjust leveling nuts and plates to correct elevation and verify
4. thread cap over threaded rods onto plates
5. form around bedding layer
6. grout bedding through grout port
7. grout ducts through ducts at cap top, if plates block grout flow
8. tighten bolts

2.2.4.3 IRC Review Topics

Committee comments focused on reduced grouting operations, construction sequences, and a "grout-less" detail. The Pierce Street Elevated project successfully used bolted connections, but required three stages of grouting: 1) bedding layer through grout ports in the cap sidewall; 2) vertical ducts from the cap top; and 3) cap top anchorage recesses. A reduction to one grouting operation was recommended, as listed in the first construction sequence. This could be achieved by combining grouting of the sheathing ducts and bedding layer into one operation and moving the anchorage to the slab (Figure 2.13), thereby eliminating the recessed pockets. Eliminating recesses would also alleviate congestion of cap top reinforcement and eliminate any durability concerns associated with the recesses. The committee also agreed that a provision for an exit port was essential to ensure adequate grouting of the bedding layer.

Two alternative construction sequences were discussed, both of which would require anchoring connectors into the column tops after placement of the cap. One approach involved threading the bolts through the ducts into threaded anchors located at the top of the columns. The other approach, similar to that mentioned in Section 2.2.2.2, used the cap as a template to drill anchorage holes into the column tops, then required placing resin capsules in the holes and setting the bolts through the cap into the holes to activate the resin. The former option was considered more problematic because little recourse would exist if threading or tolerance problems arose in the field.

Details of a "grout-less" connection were introduced and the pros and cons weighed. In addition to eliminating grouting concerns, a main advantage would be the extremely short time to beam setting. An important feature of this detail is the use of a neoprene bearing pad at the cap-to-column interface to accommodate tolerances. After grouting the bolts and placing the pad, the cap would be threaded over the bars and the bolts tightened to complete the connection. Despite apparent advantages, much concern was expressed over pad deformability, strength, fabrication, durability, and especially the need to accommodate tolerances, super elevation and slope. In addition, grout might still be required to provide corrosion protection for the bolts in the ducts. It was agreed that a grout-less connection would be difficult to practically implement.

2.2.4.4 Disadvantages and Uncertainties

Similar to grouted vertical ducts, there are few obvious disadvantages for bolted connections. Disadvantages include:

- special durability protection required for the cap top anchorage
- potentially complicated grouting operations for the bedding and ducts (leveling nut option)

Uncertainties include:

- performance compared with monolithic construction
- failure modes
- gravity-flow grouting of multiple ducts through the bedding layer
- grout performance requirements
- influence of bedding layer shims on connection behavior
- cracking at bedding layer under service level loads
- influence of bolts on grouting operations
- special reinforcement requirements under cap top anchorage plates

2.3 CONNECTORS

As shown in Section 2.2, a variety of connectors may be used in precast bent cap connections. This section describes in further detail several connector options, including: 1) straight reinforcing bars, 2) headed reinforcing bars, 3) hooked reinforcing bars, 4) U-shaped reinforcing bars, and 5) bolts. Epoxy coating of connectors is also briefly introduced.

2.3.1 Straight Reinforcing Bars

The simplest option for precast connectors is straight reinforcing bars. The most common bars are ASTM A615 Grade 60 reinforcing bars, which are inexpensive, readily available, non-proprietary, and have well-defined properties. These bars also accommodate horizontal tolerances more easily than other connector options. However, straight bars are rarely used in precast connections due to expected large development lengths. If the ACI 318-99 development length requirements for straight bar anchorage in cast-in-place concrete were used for connectors, connectors would be limited to #6 bars or smaller for rectangular caps having a depth of 42 in. or less [2.6, Section 12.2]. However, it is preferable to use connectors in the range of #9 and #11 rebars commonly used in bridge columns. Piles are expected to use smaller diameter bars.

Due to the availability of large cover for connectors, as well as the use of steel ducts and/or confining reinforcement for connections, the required development length is expected to be less that that required by ACI 318-99. Limited available literature suggests significant reductions in development length for confined connectors. Compared to the ACI 318-99 development length requirement, the PCI Design Handbook indicates approximately a 40% reduction in development length for straight bars as large as #8's embedded in grout-filled steel ducts with a side cover of at least 3 in. [2.7, Section 6.5.1]. Phase 1-3 tests will help establish requirements for straight bars embedded in grout pockets and vertical ducts. Grouted sleeve couplers use similar principles of confinement to develop rebars in short sleeves.

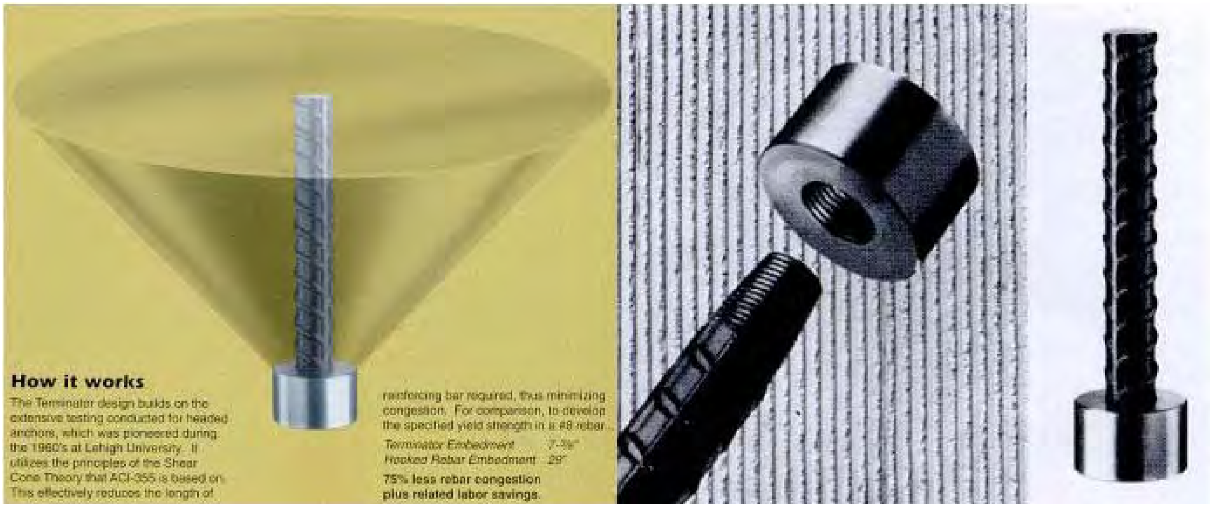
The preceding discussion applies to the development of connectors within the precast bent cap. Connectors must also be anchored adequately into piles or columns. Due to the relatively large available pile and column lengths, developing this end of the connector can be easily achieved with cementitious grouts or adhesives. Epoxy or polyester grouts may be preferred for use in piles or columns because of the quicker curing time (often less than 20 minutes). In either case, available embedment depths should be sufficient for the tensile strength of connectors to be developed.

2.3.2 Headed Reinforcing Bars

Headed reinforcing bars are expected to require shorter development length requirements than straight bars. Headed bars achieve improved anchorage by using bearing at the head in addition to bond as a load-transfer mechanism. Grout pocket and grouted vertical duct connections are appropriate applications for headed bars.

Figures 2.15 and 2.16 show three types of proprietary headed bars that may be used in precast bent cap systems: the Lenton *Terminator* and Headed Reinforcement Corporation's T-headed bar and upset-headed bar. The Lenton *Terminator* is an oversized coupling secured to the taper-threaded end of a rebar. The threads provide a self-aligning, positive lock system that is easily engaged in the field with 4.5 turns [2.8]. With this head, a #9 bar in 4000 psi concrete can be developed in just 10 in., and a #11 bar in 12 in., less than 20% of the standard development lengths. This performance may be a significant advantage for some connections.

In addition to development length considerations, construction tolerances and cost are vital factors in determining the value of a headed bar. As shown in Table 2.2, the *Terminator* head diameter for a #9 bar is 2.25 in., or $1.5d_b$. This corresponds to a horizontal tolerance of 0.9 in. for a 4-in. duct, and a 1.4-in tolerance for a 5-in. duct. The 3-in. diameter head used with a #11 bar would necessitate the use of a 5-in. duct to provide a 1-in. tolerance. This type of headed bar, therefore, could not be used if larger tolerances were required for #11 bars in vertical ducts. Grout pocket connections, however, could be sized to accommodate the larger head. The cost of the head and taper for a #9 bar is approximately $10/bar, a minor cost overall. It is evident that the Lenton *Terminator* could be useful for some applications.



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 7-⅜"
Hooked Rebar Embedment 29"

75% less rebar congestion plus related labor savings.

Figure 2.15 Lenton *Terminator* [2.8]

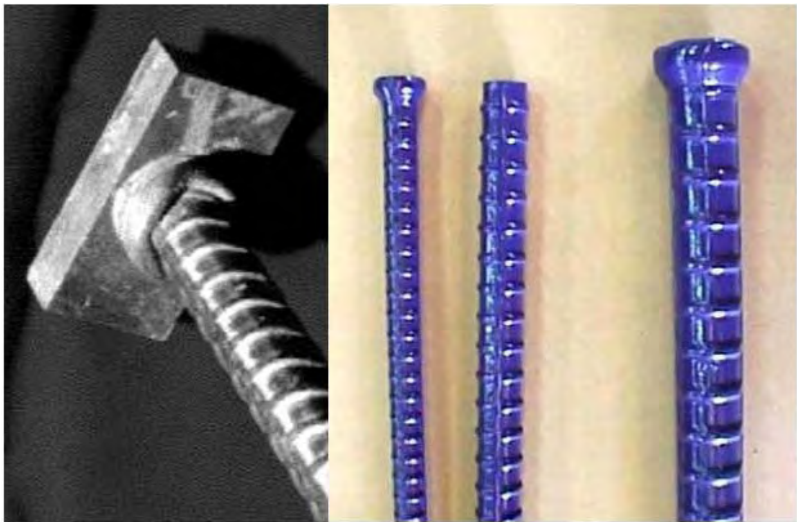


Figure 2.16 T-headed bar and Upset-headed bars [2.11,2.12]

Table 2.2 Lenton *Terminator* Properties [2.8]

Rebar Size	Head Diameter (in)	Head Thickness (in)	Development Length (in) f'c=4000 psi
#8	2.25	1.38	8.5
#9	2.25	1.50	9.7
#11	3.00	1.69	12.0

Headed Reinforcement Corporation (HRC) has recently developed T-headed bars that use steel plates welded to a reinforcing bar by a carefully controlled friction-forging process, according to ASTM A970 (Figure 2.16, [2.9]). Although development along the reinforcing bar exists, T-heads are designed to develop the bar strength by bearing alone. This would permit short connector lengths for precast bent cap systems. T-heads are available in a variety of head shapes, including round heads, which maximize horizontal tolerances. The T-head diameter for a #9 bar is 2.75 in., or approximately $2.4d_b$. This provides a tolerance of 0.6 in. for a 4-in. duct and 1.2 in. for a 5-in. duct. The 3.4-in. head used with a #11 bar provides approximately 0.75-in. tolerance with a 5-in. duct. It is evident that even with 5-in. ducts, sufficient construction tolerances may not be available for grouted vertical duct connections, although grout pockets could be sized appropriately. The cost of T-heads is approximately $8/bar for a 10 ft. length, again a minor cost overall [2.10]. Devries documents extensive pullout tests of T-headed bars [2.11].

HRC has also recently developed upset-headed bars, as shown on the right side of Figure 2.16 [2.12]. Originally developed to aid in coupling of bars, upset-headed bars may be a reasonable solution to the potentially large development length requirements of straight bars and the minimal construction tolerances available for T-heads. Upset-headed bars use a head with a diameter of $1.4d_b$. After heating the bar with a torch, the upset head is forged onto the bar end in the field using HRC's "Upsetter" (Figure 2.17). Little is know about the anchorage abilities of upset-headed bars at present, but anchorage performance is likely to be between that of straight bars and T-heads.

Figure 2.17 "Upsetting" Process [2.12]

Reasonable horizontal tolerances and cost are expected for upset headed bars. For a 4-in. duct, a #9 and #11 upset-headed bar would provide a tolerance of approximately 1.25 in. and 1 in., respectively. An additional 0.5 in. would be available for a 5-in. duct. Unit costs for upset-headed bars are unavailable; however, the main expense is the portable "Upsetter," which currently costs approximately $100-$200/month for rental [2.10]. For a typical project, the unit cost of an upset-headed bar should not exceed that of a T-head.

2.3.3 Hooked Reinforcing Bars

Hooked reinforcing bars are an option for grout pocket connections. Bearing on the inside of a hook provides additional anchorage that results in a development length significantly smaller than that of a straight bar in cast-in-place concrete. In fact, the use of a hook reduces the required development lengths for #9 and #11 bars a sufficient amount to be applicable in precast bent cap systems. Like straight bars, this connector option is inexpensive, readily available, non-proprietary, and has well-defined behavioral characteristics and design requirements for cast-in-place concrete.

The main challenge is satisfying horizontal construction tolerances. A standard 180-degree hook would fit more easily within a grout pocket than a 90-degree hook. As shown in Figure 2.18, a #9 bar and #11 bar with a 180-degree hook would require a minimum pocket width of about 11 in. and 13 in., respectively, plus tolerances. If hooked bars are used in grout pockets similar to those shown in Figures 2.1 and 2.2, this requirement would not be excessive. However, the placement of connectors near the center of a column would require unreasonably large pocket sizes. Because of this, the maximum number of connectors in a pocket would be limited.

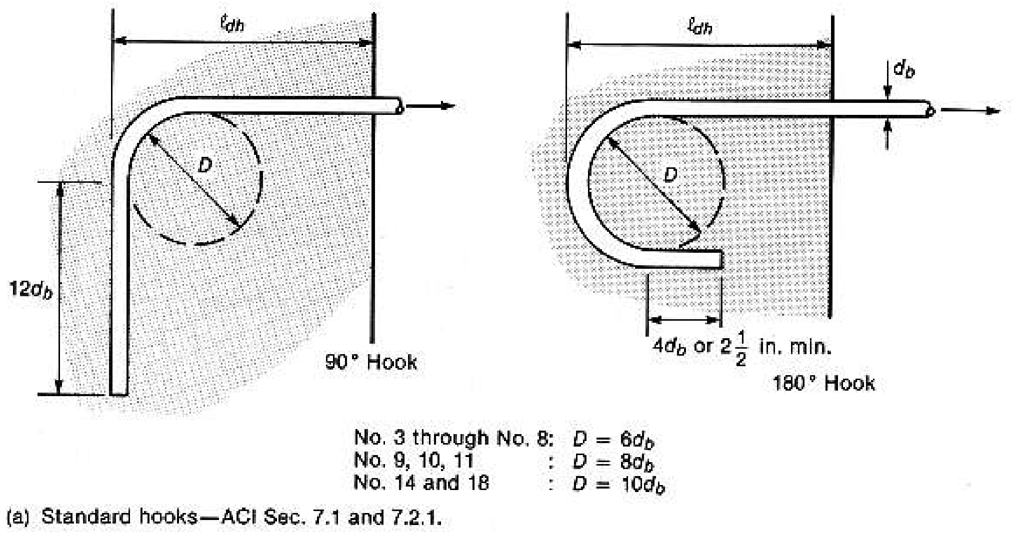


Figure 2.18 Hooked Bars [2.13]

2.3.4 U-shaped Reinforcing Bars

U-shaped reinforcing bars for grout pocket connections (Figure 2.1) are an option similar to hooked bars. U-shaped bars were successfully implemented in Red Fish Bay grout pocket connections. In that case, standard #9 reinforcing bars were bent 180 degrees into U-shapes and then grouted into embedded sleeves in the piles. Since both ends of a U-shaped bar must be embedded into a column or pile, development of the bar in the connection region is not of concern. This is the primary advantage over hooked bars. However, U-shaped bars must be carefully bent to properly fit into pile sleeves. For cast-in-place columns, placement is critical for both U-shaped bars and hooked bars. Like straight and hooked bars, U-shaped bars are inexpensive, made from readily available straight bars, and non-proprietary.

Similar to hooked bars, the main challenge for U-shaped bars is satisfying horizontal construction tolerances. The same minimum pocket widths for #9 and #11 hooked bars apply to U-shaped bars, and the same concern over placement of connectors near the center of a column exists.

2.3.5 Bolts

Bolted connections for precast bent cap systems use threaded rods or post-tensioning bars, referred to herein as bolts, that extend through the depth of the cross section and are anchored at plates on the top of the cap. Threaded rods are easily anchored at the cap top. Bond along the threads provides additional anchorage and redundancy. A large variety of both non-proprietary and proprietary bolts are available for this application.

An example of a non-proprietary bolt is threaded rod. Threaded rod uses threads of standard size and pitch together with standard bolts, nuts and washers. For example, ASTM A193 B7 defines a 105 ksi yield strength for bolt diameters up to 2.5 in., a substantially larger strength than ASTM A615 Grade 60 rebar. Use of this higher strength material may enable a bolted connection to use fewer connectors than required by standard reinforcing bars. This may be a useful advantage for cases in which a large connector area is required.

Many proprietary bolts are also available. Dywidag *Threadbar* reinforcing steel is commonly used in U.S. construction [2.14]. *Threadbar* uses a continuous rolled-in pattern of thread-like deformations that allow special nuts and couplers to be threaded along the length. *Threadbar* conforms to ASTM A615, has properties similar to standard reinforcing bar, and is available in Grade 75 for bar diameters over 1 in.

Dywidag also has a post-tensioning system that uses high strength steel bars (Figure 2.19) with strengths of 150-160 ksi. These hot-rolled bars conform to ASTM A722 and are available in diameters appropriate for precast bent cap systems: 1 in, 1-1/4in, and 1-3/8in. As shown in Figure 2.20, this system also features bearing plates, sheathing ducts, grout tubes, and pocket formers, all of which may be useful in a bolted connection.

Williams produces *All-thread Rebar* and *All-thread* post-tensioning bar similar to Dywidag products (Figure 2.21, [2.15]). As shown in Figure 2.22, one difference between the products is that *All-thread Rebar* uses a 360-degree concentric thread deformation pattern that uses a high relative rib area, leading to excellent bond. *All-thread* conforms to ASTM A615, is available in Grade 75 (standard) and Grade 60, and is fabricated in diameters for most rebar except #9's. Williams *All-thread* post-tensioning bar is available in Grade 150 and conforms to ASTM A722.

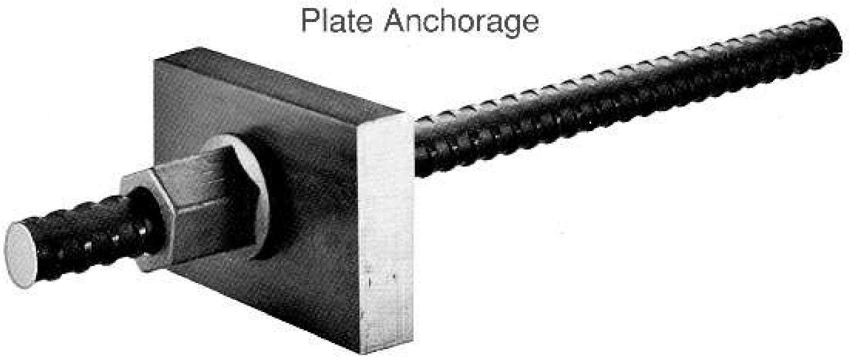


Figure 2.19 Dywidag *Threadbar* and Plate Anchorage [2.14]

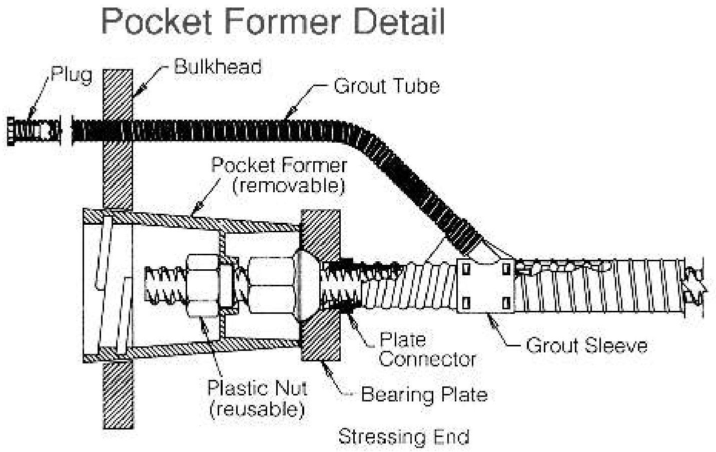


Figure 2.20 Dywidag *Post-Tensioning System* Pocket Former Detail [2.14]

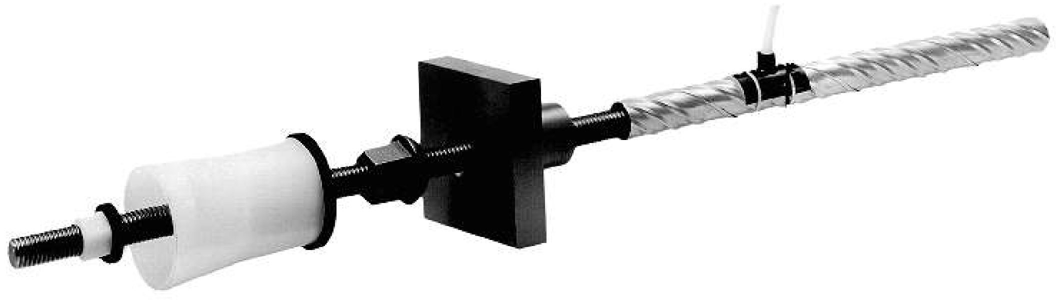


Figure 2.21 Williams *Post-Tensioning System* [2.15]

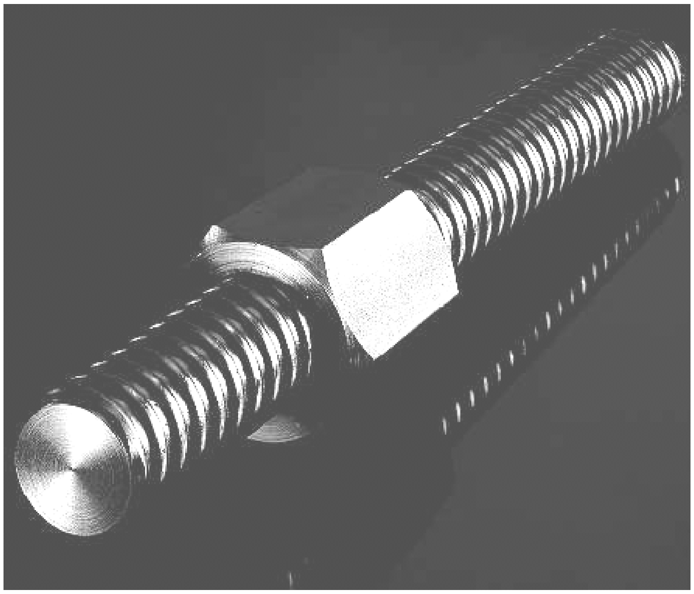


Figure 2.22 Williams *All-Thread Post-tensioning Bar* [2.15]

2.3.6 Epoxy Coating

To enhance corrosion protection, epoxy-coated bars are commonly specified for bridge projects in conformance with AASHTO specifications [2.16, Division II]. Epoxy-coated U-shaped bars were used for the Red Fish Bay precast bent cap system, as shown in Chapter 1. Epoxy coating is available not only for straight and bent deformed reinforcing bars, but also grouted sleeve couplers (Figure 2.23) and bolts. New advances are continually being made in epoxy-coating technology. For example, the upset-headed bars shown in Figure 2.16 use the 3M Scotchkote 426 coating, a relatively new fusion-bonded epoxy coating that provides better resistance to disbondment and corrosion, especially in marine environments [2.17]. Because bond strength is normally reduced for epoxy-coated bars, the research team decided to use epoxy-coated bars in Phase 1-3 tests. Other type of protective coatings such as galvanized steel or stainless steel may also be used for bolts and anchorage hardware.

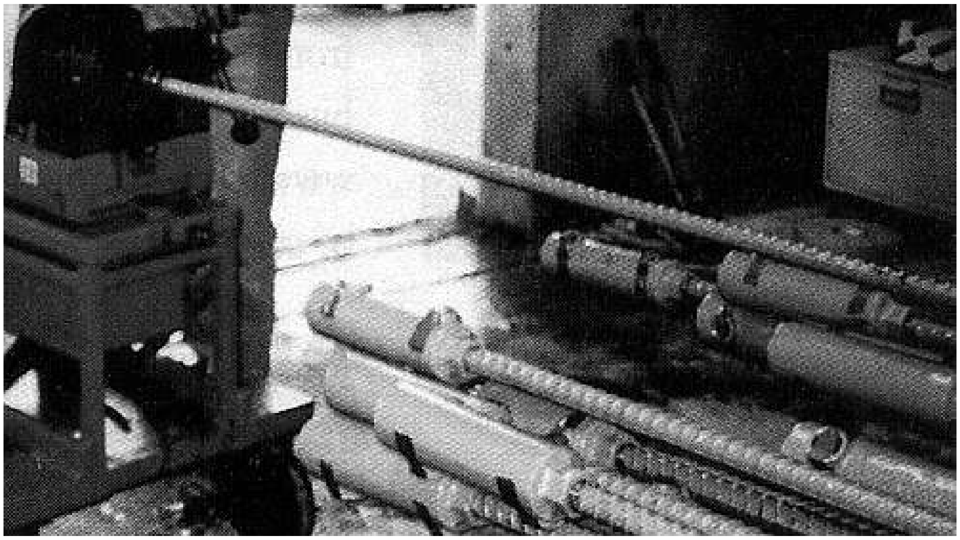


Figure 2.23 Epoxy-Coated Grouted Sleeve Couplers and Rebars [2.3]

2.4 GROUT

The successful implementation of precast bent cap connections vitally depends on the performance and placement of grout within connections. In bridge projects where grouting has been used, grout problems are usually traced to three problems: 1) poor details, 2) insufficient grout performance, and 3) inconsistent grouting procedures [2.18]. While previous sections have addressed connection details, this section discusses the development of a grout specification and acceptable grouting operations.

2.4.1 Grout Specification

A suitable grout specification is considered an integral part of a precast bent cap system. Development of a specification requires that appropriate criteria be established and that standard tests be identified to assess the performance of the grout in achieving these properties. In addition, because it is expected that prepackaged grouts will be used, several readily-available, proprietary grouts must be identified for use. To validate the adequacy of this specification, construction of specimens and test results from Phases 1-3 are required.

This section discusses the components of a grout specification and the underlying rationale used in development of the specification. After an introduction to the ASTM specification for non-shrink grout, components of the specification are discussed according to the following four categories: mechanical properties, compatibility, constructability, and durability. The properties of three readily available grouts are then compared to this specification. In the final section, the advantages and disadvantages of extended grouts are considered, and results from trial batches are discussed.

2.4.1.1 Non-Shrink Grout

Grout is expected to be a suitable material for completing precast bent cap connections. Typically, grout consists of a mixture of water, fine aggregate, cementitious material, and admixtures. Grout without coarse aggregate extension is usually referred to as neat grout. Cementitious grouts have been used more in precast construction than grouts such as epoxy or polymer-modified grouts. Epoxy or polymer-modified grouts can have significant advantages, such as a high strength in a shorter time (e.g., 6 ksi in 6 hours), better bond, reduced chloride permeability, improved freeze thaw durability, and lower creep. However, they are often significantly more expensive and less compatible with surrounding concrete. In addition, if the resin is used in too large a volume, the heat of reaction may cause it to boil, and thereby lose strength and bond. The approach adopted in developing precast bent cap systems is use cementitious grouts as a first priority, with other grouts as secondary options.

A primary disadvantage of cementitious grouts is the shrinkage and cracking that result from the use of hydraulic cement. Non-shrink grout compensates for this shrinkage by incorporating expansive agents into the mix. With non-shrink grout, the effects of shrinkage cracks or entrapped air on the transfer of forces and bond are minimized, though not eliminated. ASTM C 1107 establishes strength, consistency, and expansion criteria for prepackaged, hydraulic-cement, non-shrink grout. Tables 2.3-2.5 list the performance criteria found in ASTM C 1107. This specification, which applies to neat grouts, is a useful starting point in discussing a specification for precast bent caps. However, additional properties are required to achieve the desired grout performance for a precast bent cap system.

Table 2.3 ASTM C 1107 Compressive Strength Requirements [2.19]

Age	Compressive Strength Requirement psi
1 day (optional)	1000
3 days	2500
7 days	3500
28 days	5000

Table 2.4 ASTM C 1107 Consistency Options [2.19]

Consistency Options: ASTM C 939 flow cone or ASTM C 109 flow table			
Flow Classification	Plastic (ASTM C 109)	Flowable (ASTM C 109)	Fluid (ASTM C 939)
Plastic and Flowable			
Maximum at 5 drops	125%	<150%	N/A
Maximum at 5 drops	100%	>125%	N/A
Fluid			
Maximum, seconds	N/A	N/A	30
Minimum, seconds	N/A	N/A	10

Table 2.5 ASTM C 1107 Expansion Requirements [2.19]

Expansion Requirements: 3 x 6 in. vertical height change cylinders			
	Grade A Prehardening (ASTM C 827)	Grade B Post hardening (ASTM C 1090)	Grade C Pre- & Post hardening (ASTM C 827 & C 1090)
Early height change			
Maximum % at final set	+4.0	N/A	+4.0
Minimum % at final set	-0.0	N/A	-0.0
Height change of moist-cured hardened grout at 1, 3, 7, 14, and 28 days			
Maximum %	N/A	+0.3	+0.3
Maximum %	-0.0	-0.0	-0.0

2.4.1.2 Mechanical Properties

Table 2.6 lists the properties required by the grout specification, as well as applicable ASTM specifications and a range of acceptable values for each property.

Table 2.6 Precast Bent Cap Grout Specification

Property	Values	
Mechanical	Age	Compressive strength (psi)
Compressive strength (ASTM C-109, 2" cubes)	1 day	2500
	3 days	4000
	7 days	5000
	28 days	5500
Compatibility		
Expansion requirements (ASTM C 827 & ASTM C 1090)	Grade B or C—expansion per ASTM C 1107	
Modulus of elasticity (ASTM C-469)	3.0-5.0×10^6 psi	
Coefficient of thermal expansion (ASTM C-531)	3.0-10.0×10^{-6}/deg F	
Constructability		
Flowability (ASTM C-939)	fluid consistency efflux time: 10-30 seconds	
Set Time (ASTM C-191)		
Initial	3-5 hrs	
Final	5-8 hrs	
Durability (as necessary)		
Freeze Thaw (ASTM C-666)	300 cycles, RDF 80%	
Sulfate Resistance (ASTM C-1012)	expansion at 26 weeks < 0.1%	

Using the same simplicity of ASTM C 1107, mechanical property requirements for a precast bent cap system are defined by a minimum compressive strength. In addition to compressive strength, tensile strength is importance to connection behavior. It is expected that tensile strength for grouted bars may be taken proportional to the square root of the compressive strength.

Adequate compressive strength gain and final strength are necessary to transfer connection forces at different load stages (e.g., beam setting and service level). As shown in Table 2.4, ASTM C 1107 defines minimum compressive strength requirements at 1 day, 3 days, 7 days, and 28 days [2.19]. While these values may be adequate in some cases, the specification shown in Table 2.6 defines higher minimum values. Especially important are the 1-day strength of 2.5 ksi and 3-day strength of 4 ksi, which should enable contractors to expedite construction.

Higher compressive strength requirements also help prevent grout from being the weak link in the system. Grout strength should exceed that of the surrounding concrete. A New Zealand precast concrete committee suggested grout strength should exceed the concrete strength by 1.4 ksi [2.20]. The TxDOT Standard Specification requires Class C concrete to achieve a 28-day strength of at least 3.6 ksi [2.21]. In the field the strength often exceeds 5 ksi at 28 days. In addition, some caps use concrete with a specified compressive strength of 5 ksi or more at 28 days. For the grout strength to exceed a 5-ksi concrete at 28-days, a grout cube strength of 6.3 ksi would be required. This assumes the commonly-used 0.8 factor for converting cube strength to cylinder strength is adopted. Many proprietary grouts satisfy this or even higher compressive strength criteria.

2.4.1.3 Compatibility

Compatibility is a measure of the similarity of properties between the grout and surrounding concrete. For the grout specification, compatibility refers to the following grout properties: volume stability, modulus of elasticity, and coefficient of thermal expansion. The importance of these properties for a given connection depends in large measure on the materials surrounding the grout as well as the function of the grout. For example, for grout pocket connections, all three properties are important, since the grout is surrounded by the cap concrete and an important function of the grout is to transfer loads between the concrete cap and connectors.

For grouted vertical ducts, grout is also used to transfer loads; however, in this case, load transfer is through the steel ducts. Hence, closely matching the coefficient of thermal expansion between surrounding materials has less meaning. Volume stability should still be considered for this connection, as described later. Similar issues arise for bolted connections.

With these considerations in mind, it is clear that the range of values shown in Table 2.6 for the modulus of elasticity and coefficient of thermal expansion is intended to provide a fairly close match in stiffness between the grout and concrete. Normal weight concrete has a modulus of elasticity of approximately 4×10^6 psi for a compressive strength of 5 ksi, and a coefficient of thermal expansion of about 6×10^{-6}/degree F. Most proprietary grouts easily fit within the ranges shown in Table 2.6.

The performance requirements for volume stability are more complicated, since ASTM C 1107 allows three grades of shrinkage compensating grouts (Table 2.5): Grade A—prehardening volume-controlled type, Grade B—post-hardening volume-controlled type, and Grade C—combination volume-controlled type. Shrinkage is determined using two ASTM specifications: C 827 for prehardening grout and C 1090 for post-hardening grout. Both rely on vertical height changes of 3 x 6 in. cylindrical specimens to determine settlement shrinkage. Drying shrinkage is precluded.

It is expected that grade classification may influence the performance of a connection. Since Grade A grouts can produce as much as a 4-percent volume expansion before the grout hardens, a reduction in density of the hardened grout may result, as well as shrinkage stresses larger than for a Grade B or Grade C grout. Because Grade B and Grade C grouts expand after hardening, precompression may develop in the connections. This is expected to enhance connection performance. For these reasons, Table 2.6 shows Grade B and Grade C options for a precast bent cap system, with the same limits on volume change as stipulated in ASTM C 1107. Phase 1-3 testing will provide insight into the influence of the grade on connection performance.

2.4.1.4 Constructability

The two constructability requirements for grout are flowability and set time. An IRC member once stated, "Constructability is providing all the engineering properties you want with the simplest construction operations." In this light, the importance of grout consistency cannot be overemphasized, for simplicity in grouting operations hinges mainly on grout flowability. Grouts should be used in a fluid consistency, with an efflux time, or flow, between 15 and 30 seconds as determined by the Flow Cone Method defined in ASTM C 939. This flow time should ensure that the grout displaces all air within the bedding layer and pocket or duct voids, that air bubbles easily rise to the surface, and that the grout bonds well within the connection. It is expected that a flow time shorter than 15 seconds may be lead to segregation of the grout. A longer flow time may inhibit complete filling of the voids and may entrap air.

Most technical data sheets indicate the amount of water required for fluid consistency; however, this amount usually applies only under certain temperature conditions. Water, of course, cannot be added indiscriminately to the mix to increase flow, as this may adversely affect strength and cause bleeding or segregation. Depending on temperature, ice or warm water may be used to adjust the flow. Hence, to satisfy this provision, contractors need to mix trial batches of grout for specific field conditions and to use a flow cone. While this is not a procedure commonly used in field grouting, the IRC agreed that such an approach is both necessary and reasonable.

Set times, related to hydration between cement and water, should be sufficiently short to enable construction operations to proceed quickly. The required range of initial and final set times are shown in Table 2.6. Set times for most grouts easily fall within this range.

A final property related to constructability is the grout working time. The working time, or pot life, is the amount of time a grout remains workable. A sufficiently long working time is crucial for the successful implementation of the system, especially as contractor experience in grouting precast connections will initially be limited. If the working time is too short, inadequate placement or consolidation may result, leading to voids in the connection and loss of contact area in the bedding layer. In addition, short working times encourage retempering. This should be avoided, as it may affect properties such as strength and durability.

Proprietary grouts typically require grout be placed and tamped within 15-30 minutes. It is expected that at least one connection may be grouted within this time frame. However, the grout specification does not include a minimum working time for the grout, because the time required to grout a connection will vary according to specific application, grouting approach, and contractor experience. Establishing a required working time may also unfairly eliminate certain proprietary grouts from consideration.

2.4.1.5 Durability

Designing connection grout for durability is an important factor in connection durability. Durability of concrete is often determined by assessing various properties such as the water-to-cement ratio, permeability (ASTM C 1202), absorption (ASTM C 642), freeze-thaw (ASTM C 666), and sulfate resistance (ASTM C 1012). Grout manufacturers, as well as certain guide specifications, recommend the same ASTM specifications to assess grout properties. To enhance durability properties, manufacturers may include one or more of the following: chemical admixtures (water reducers, super plasticizers), mineral admixtures (fly ash, silica flume), air entraining agents, corrosion inhibitors, and fibers [2.22].

Grout durability should be at least equal to that of the surrounding concrete, and proprietary grouts are often formulated to achieve this. For this reason, required values for permeability or absorption are not included in the specification. However, other properties may be required on a project-specific basis to assess durability, such as freeze-thaw and sulfate resistance. Testing may be necessary if properties are not readily available from the manufacturer. Freeze-thaw resistance may be determined using ASTM C 666. ASTM C 1012 may be used to determine sulfate resistance, an important property for some coastal regions where exposed grout surfaces may be located in the splash zone of water containing sulfates. Table 2.6 shows required properties for freeze thaw and sulfate resistance of grouts. Additional properties related to durability such as scaling resistance (ASTM C 672) and resistance to deicing salts (ASTM C 672) should be investigated as necessary. Grouts should also be chloride-free.

Strictly speaking, bleeding is not a property directly related to durability. However, bleeding represents a form of segregation that could adversely affect connection durability. In the context of a precast bent cap connection, bleeding is the development of a layer of water at the top of a grout pocket or duct due to the settlement of solids (cement and sand) coupled with an upward migration of water. Excessive bleeding may result in a top surface with a high water-cement ratio, resulting in poor durability, reduced strength, and even water pockets [2.23]. Of equal concern is the potential for a pocket to form at the top of a connection after the evaporation of bleed water. Water may later collect in a pocket and seep into the connection. Non-shrink grouts normally use low water-cement ratios to achieve sufficient strength, durability, and set times, and to limit shrinkage. In addition, they are formulated to have little to no bleeding. For these reasons, a limitation on bleeding is not included in the specification, although bleeding of grouts is examined in Phase 1-3 tests.

2.4.1.6 Comparison of Proprietary Grouts

Three manufacturers for grouts commonly used in Texas were contacted to verify the availability of specific hydraulic cement-based grouts that might satisfy the grout specification. These manufacturers are Master Builders Technologies, Sika Corporation, and the Euclid Chemical Company. After manufacturers reviewed connection details, they suggested one of their grouts for application.

Table 2.7 lists the suggested grouts and grout properties [2.24-2.26]. Although required compressive strengths are considerably higher than that required by ASTM C 109, all data specifications, shown in Table 2.7 as well as Appendix A, appear to satisfy the strength requirement. An approximate cost per bag is: Sika 212—$7; Euclid High Flow—$17; MF928—$19.

Compatibility properties also fall well within acceptable ranges. The modulus of elasticity data available for two of the three grouts correspond reasonably well with that of concrete, as does the coefficient of thermal expansion (available only for Masterflow 928). As required by the specification, all grouts are classified as either Grade B or Grade C grouts for volume stability.

Flowability and set times are also expected to be suitable for application. All grouts may be used in the fluid consistency (15-30 second flow), and property values listed in Table 2.7 correspond to this consistency. The working time for each grout, however, differed considerably. Masterflow 928 (MF928) and Euclid High Flow (EHF) may provide a working time of approximately 30 minutes, and Sika 212 only 15 minutes. This is obviously an important grouting consideration.

Table 2.7 Comparison of Grout Specification with Prepackaged Grouts

Property	Specification	Masterflow 928	Sika 212	Euclid High Flow
Mechanical				
Compressive strength (ASTM C-109, 2" cubes)	Compressive strength, psi 2500 4000 5000 5500	3500 4500 6500 7500	2700 * 5500 5800	4000 6000 7000 9000
Compatibility				
Expansion requirements (ASTM C 827 & ASTM C 1090)	Grade B or C—expansion per ASTM C 1107	Grade B 0.02-0.03% at 28 days	Grade C 0.027% at 28 days	Grade C 0.07% at 28 days
Modulus of elasticity (ASTM C-469)	$3.0\text{-}5.0\times10^6$ psi (28-days)	$3.0\text{-}3.9\times10^6$ psi, min	*	*
Coefficient of thermal expansion (ASTM C-531)	$3.0\text{-}10.0\times10^{-6}$/deg F	7.5×10^{-6}/deg F, max	*	*
Constructability				
Flowability (ASTM C-939)	fluid consistency efflux time: 15-30 sec	fluid consistency efflux: 25-30 sec	fluid consistency efflux: 10-30 sec	fluid consistency efflux: 16 sec
Set Time (ASTM C-191)				
Initial	3-5 hrs	4 hrs	4.5-6.5 hrs	4 hrs
Final	5-8 hrs	6 hrs	6-8 hrs	5 hrs
Working Time		30 minutes (45–90 deg F)	15 minutes (65–75 deg F)	30 minutes (60–70 deg F)
Durability				
Freeze Thaw (ASTM C-666)	300 cycles, RDF 80%	300 cycles, RDF 90%	"superior"	*
Sulfate Resistance (ASTM C-1012)	expansion at 26 weeks < 0.1%	0.03%	*	*

Few durability properties were available for the recommended grouts, although each manufacturer mentioned that durability would be equal to or greater than that of similar strength concrete. As shown in the table, satisfactory freeze thaw performance would be expected for MF928 and Sika 212. Master Builders also provided a copy of test data [2.27] to verify that MF928 provides outstanding sulfate resistance (0.03% expansion at 26 weeks, per ASTM C 1012). All products are chloride-free. Durability provisions may be advisable if they result in other desirable properties.

2.4.1.7 Extended Grout

2.4.1.7.1 Advantages and Disadvantages

Substantial consideration was given to the use of extended grout instead of neat grout, due to the following potential benefits of extended grout:

1. more compatible with concrete
2. better interlock between connection components
3. denser, less permeable
4. less drying shrinkage
5. less creep
6. better heat sink
7. larger grout volume per bag, hence less expensive
8. better link to data for cast-in-place anchors

Some manufacturers recommend extending their grouts, typically with 1/2-in. or 3/8-in. coarse aggregate, when filling larger voids. Aggregate should be clean, well-graded, saturated surface dry, have low absorption and high density. The usual reasons manufacturers give for extending grouts are: 1) to provide cost savings by reducing the volume of grout required, 2) to provide better compatibility with concrete, and 3) to reduce drying shrinkage. Compatibility and force transfer considerations may be prime advantages for precast connections. The cost savings in grout material are expected to be negligible and more than offset by the constructability concerns for extended grout, as mentioned below. Durability advantages are unlikely to be significant, given the normally adequate durability provided by neat grout.

On the other hand, the use of extended grout presents numerous disadvantages, such as:

1. poorly defined properties
2. less certain quality control related to aggregates, mixing, flowability, and consolidation in the bedding layer and other voids
3. unclear responsibility related to the quality of the grouted connection
4. additional field operations
5. more costly connections

Property requirements for extended grouts are not as well defined by ASTM specifications, which normally apply to either neat grouts or concrete. It is reasonable to apply some concrete specifications, such as that for freeze-thaw (ASTM C 666), but uncertainties exist for other properties like volume stability and flowability.

There is serious concern over a possible lack of quality control when using extended grouts. For example, successful grouting of a connection is highly dependent on the flowability of the grout. A 15-30 second flow may be carefully monitored in the field by the simple use of a standard flow cone, following ASTM C 939 (Figure 2.24). However, no similar specification exists for defining and measuring an acceptable flow for extended grouts. Also, extra quality control is required in the field for extended grouts to ensure proper selection and use of aggregates, as well as adequate mixing, flow, and consolidation of the bedding layer. Furthermore, if a problem in a completed connection arises, it may not be clear who bears responsibility—the supplier of aggregate, the grout manufacturer, or the contractor. All of this reveals the more complicated construction effort—and higher construction cost—which may result from using extended grout.

Figure 2.24 ASTM C 939 Flow Cone [2.23]

It should be mentioned that ASTM specifications for preplaced aggregate concrete, ASTM C 937 through ASTM C 940, permit the use of coarse aggregate with grout and establish proper flowability in the field using the standard flow cone. For grouted connections, the preplaced aggregate approach would involve placement of coarse aggregate in the bedding layer and ducts or pockets, followed by pumping of the connection from the bedding layer upwards. This produces a more uniform and dense mix with less grout and more aggregate per unit volume. In addition, this approach has the advantages of verifying grout flow with a standard flow cone, helps ensure the connection is filled adequately, significantly reduces drying shrinkage because of the point-to-point contact of coarse aggregate, reduces settlement shrinkage, and decreases permeability [2.23]. These advantages must be weighted against the additional complexity and cost.

2.4.1.7.2 Trial Batches

Manufacturers often recommend that trial mix designs for extended grouts be conducted to simulate job conditions. Workability and compressive strength of extended grouts was examined in a preliminary way by mixing trial batches of Euclid NS (Non-Shrink) grout, initially recommended by Euclid, and Sika 212 grout with 3/8-in. pea gravel. Although Master Builders did not recommend extending its grout, except using the ASTM preplaced aggregate method, Masterflow 928 extended with pea gravel was also later examined for comparison.

To examine workability for a flowable extended grout, one gallon of water per bag was used with Euclid NS grout. Consistency of flowable (and plastic) grouts are determined by use of a flow table according to ASTM C 109 and ASTM C 230. Fluid grouts are tested using a flow cone in accordance with ASTM C 939. At a temperature of 75 degrees F, the Euclid NS grout produced a very soupy mix that would be suitable for grouting pockets. One gallon of water per bag was also required to achieve a similarly flowable grout using Sika 212, although this was categorized as fluid grout by Sika. To produce grout in the flowable category for Sika 212, 0.8 gallons of water was required. This mix turned out to be excessively stiff and difficult to place, being nearly in a plastic state.

Compressive strengths based on 4-in. cylinders are compared for the three mixes in Figure 2.25. Although the Sika flowable grout used less water than the Euclid NS fluid grout, the Euclid grout produced higher strengths. Based on a 0.8 factor for cube strength, compressive strengths for extended grouts exceeded the guaranteed neat grout strengths established by the manufacturers, and all strengths were considered acceptable for connection grout. For comparison, neat and extended MF928 is also shown in the figure. Compressive strength of extended MF928 compares closely to that of the other extended grouts. A comparison of neat and extended grouts shows that grout strengths compared closely, although neat grout strengths were slightly lower at 7 days and slightly higher after 14 days. A 0.8 conversion factor was applied to the values for neat grout.

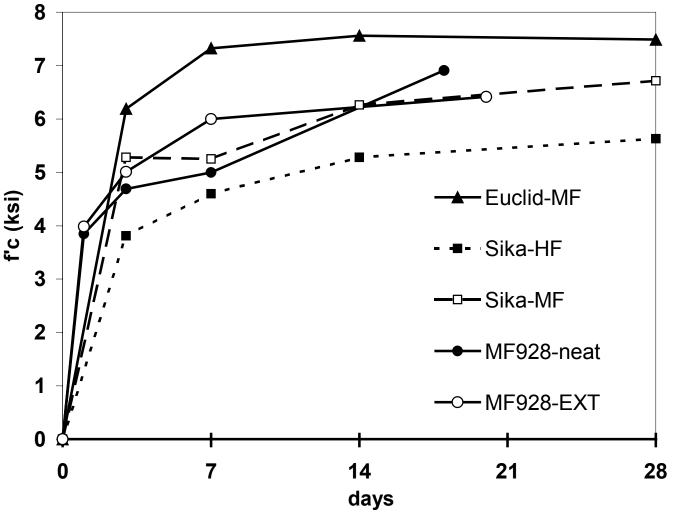


Figure 2.25 Grout Compressive Strengths for Trial Batches

2.4.1.7.3 Sika Corporation Laboratory Tests

During telephone conversations between UT and Sika Corporation, Sika suggested that either neat or extended Sika 212 grout be used for grout pocket connections, but that neat grout should be used for vertical duct and bolted connections. Because of uncertainty in extended 212 properties, UT researchers requested that Sika Corporation conduct tests on Sika 212 extended with 3/8-in. pea gravel. Sika Corporation conducted tests on fluid and flowable 212 grout extended with both 25 lbs and 50 lbs of 3/8-in. pea gravel per 50-lb bag.

Following laboratory tests, the Sika representative concluded the following [2.28]:

1. Sika 212 should be extended with no more than 30lbs of coarse aggregate per bag. Larger amounts (e.g., 50 lbs) lead to segregation of the coarse aggregate from the grout and poor workability characteristics. Mixes with 50 lbs of pea gravel were impossible to pour or trowel due to the large amount of aggregate.
2. The excessive surface area of mixes with 50 lbs of pea gravel required more cement paste than available in prepackaged bags, leading to lower strengths and poor workability.
3. Using coarse aggregate larger than 3/8-in. would reduce segregation and improve workability, compared to extended grouts with 3/8-in. pea gravel.

According to Sika data, split tensile strengths for fluid and flowable mixes and compressive strengths for flowable mixes were lowered significantly when 25 or 50 lbs of pea gravel was added. For example, the

28-day tensile strength for fluid grout extended with 25lbs of pea gravel was 1/3 lower than that found for neat grout. Greater differences existed at earlier times.

In contrast, Figure 2.26 shows that compressive strengths of extended grout in the fluid consistency were not reduced. Compared to the UT trial batch using Sika 212 (high flow), Sika laboratory strengths were higher, although beyond 14 days, there was little difference.

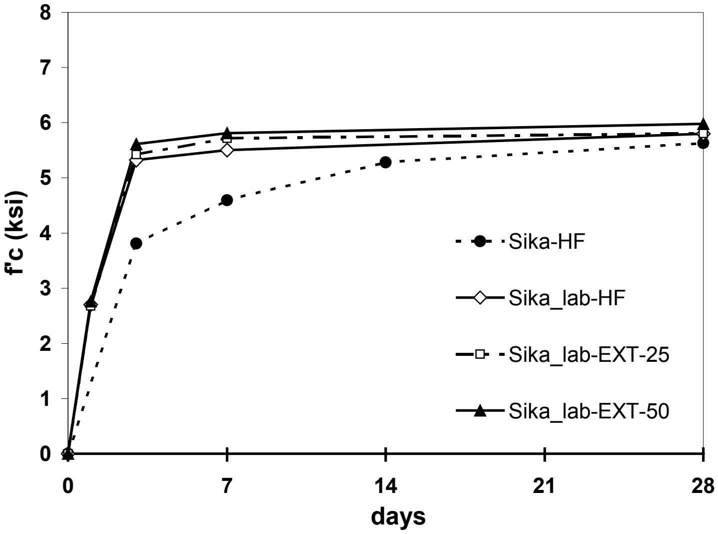


Figure 2.26 Compressive Strength for Sika 212 Neat and Extended Grouts

2.4.1.7.4 Conclusions

Based on trial batches conducted at UT and Sika, some uncertainty exists regarding the use of extended grouts. Flowability for grouts extended with 25 lbs of pea gravel appeared adequate for mixing and filling 4-in test cylinders. However, adequate flowability for grouting connections, particularly the bedding layer, was not addressed. Sika tests established that adequate compressive strength exists. However, smaller tensile strength compared to neat grout is a cause for concern. Other properties outlined in the specification were not investigated.

Following much deliberation with TxDOT engineers, the IRC, and grout manufacturers over apparent advantages and risks associated with using extended grout, the research team concluded: 1) neat grouts are preferable from a constructability and economic perspective, although with precautionary measures extended grouts may be feasible (subject to limitations outlined in Section 2.4.1.7.1); 2) proprietary mixes for neat grout are formulated with sufficient durability protection in most cases; 3) compatibility and strength issues such as connection interlock and tensile strength require further investigation. It was agreed that limited testing during Phase 1 should compare the performance of neat and extended mixes to help further address issues of flowability, compatibility, and strength.

2.4.2 Grouting Operations

This section provides further details regarding grouting operations that were briefly introduced in Section 2.2. Grouting operations must be carefully conducted as they are just as important to the implementation of precast bent cap connections as the connection details and grout.

Grouting of precast connections will normally involve post-grouting operations, i.e., grouting will be conducted after the connectors are already in place. For this case, there are two grouting approaches that are likely to be implemented: 1) gravity-flow grouting from a connection void in the cap down to the bedding layer, and 2) pressure grouting from the bedding layer upward. Gravity-flow grouting involves simpler operations overall and is expected to be less expensive. However, additional effort may be required to ensure connection voids are completely filled. Pressure grouting is more costly and requires a higher level of skilled labor in the field, but would likely result in a pocket free of voids. Both approaches may be considered options.

2.4.2.1 Gravity-Flow Grouting

The major objective in gravity-flow grouting is to completely fill the connection with grout, avoiding embedded air voids in the process. Grout mixing would be carried out in the field using an appropriately sized mortar mixer or even a hand-held mixer. Various approaches are possible for gravity-flow grouting. The following two methods are recommended for applications similar to precast bent cap connections [2.20]: tremie tube and decanting.

As shown in Figure 2.27, the tremie tube method involves lowering a flexible tube to the bottom of the void and filling the connection from the bottom upward. The tube should use a diameter at least three times the diameter of the largest size aggregate. Grout should fill the tube continuously to avoid entrapping air. The tube should remain within the grout, but may be withdrawn as the level of grout rises. It is believed that this approach is likely to fully grout a connection.

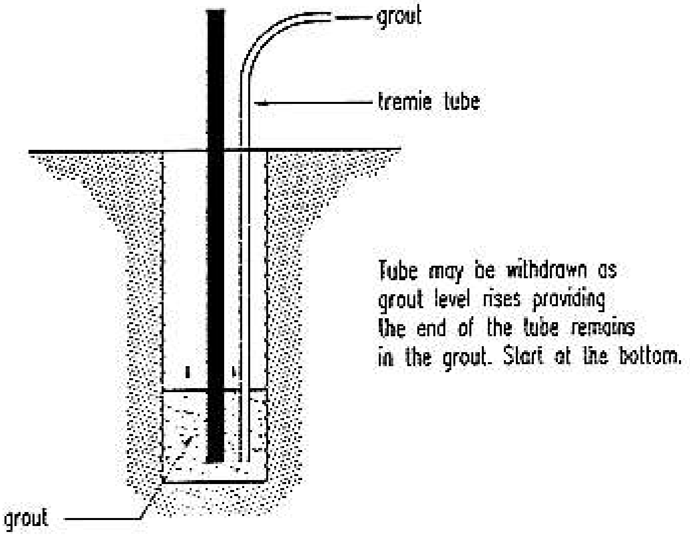


Figure 2.27 Tremie tube Grouting Method [2.20]

The decanting approach involves pouring grout against and down connectors to the bottom of the connection. A funnel and 5-gallon bucket or smaller dispenser may be used in pouring. Decanting would be feasible when a cap is placed such that adequate horizontal clearances are provided between the connector and duct or pocket. In addition, connectors would have to extend close to the cap top. This would apply to bolted connections, but would require that connectors extend beyond that required for anchorage for other connection types.

A modified tremie tube approach may be used in cases where the designer does not want connectors to extend near the top surface (e.g., due to durability concerns) or where the tube cannot extend to the bottom of a connection due to small clearances or other reasons. In this approach, a tremie tube may be used for grouting but would not be lowered to the bottom of the connection. Instead, it would remain above the level of grout, directing the flow of grout against either a connector or a sidewall. This method is expected to provide a useful alternative, but may be more susceptible than the tremie tube approach to entrapped air.

Multiple grout pockets or vertical ducts used in connections may be gravity-flow grouted from one pocket or corner duct. With a properly sealed bedding layer, grout will flow across the bedding. After the vents are progressively plugged to remove air, grout will flow up the ducts, as shown in Figure 2.28. Fluid grouts are recommended to ensure a sufficiently low viscosity to fill the ducts from beneath. For typical pocket or duct depths, sufficient pressure head should be available. After the grout level in the pocket or ducts has risen near the cap top, a tremie tube may be used to top off the openings.

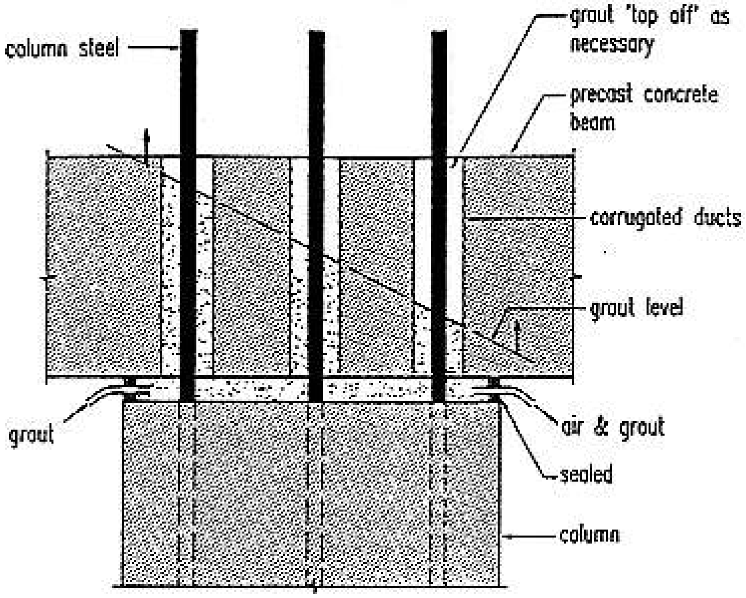


Figure 2.28 Grouting of Bedding Layer and Ducts [2.20]

Air voids within the grout or at the edge of the bedding layer are of particular concern because of the potential to adversely affect durability and force transfer. Entrapped air in vertical voids may result from one or more of the following: too high a rate of grout placement, too large of a grout viscosity, and/or non-continuous flow of grout. To help prevent embedment of air, gravity-flow grouting should be accompanied by tamping or vibrating.

Elimination of air from connections is challenging for precast bent cap connections due to the existence of both vertical and horizontal (or inclined) voids. Horizontal voids tend to trap air, especially on the horizontal shelf between the bedding layer and the precast cap. For this reason, vent tubes, or other means to vent air, should be incorporated at the bedding layer. As shown in Figure 2.29, vent tubes should be located to facilitate the removal of air. As the grout fills the void, the vent tubes flow grout in sequence. Once grout flows out of a tube without air bubbles, then the vent is plugged. This is repeated for each tube until the entire bedding is filled.

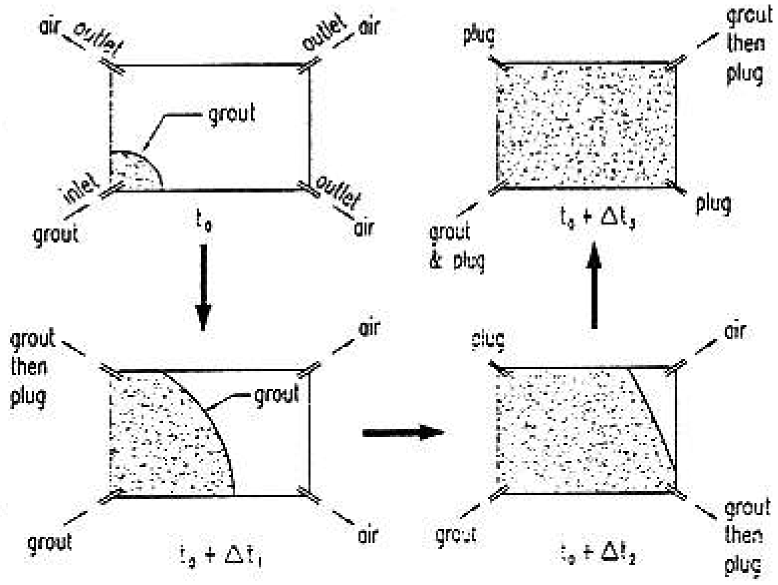


Figure 2.29 Stages for Grouting of Bedding Layer [2.20]

Due to the nature of precast connections, complete grouting of connections is difficult to establish. One approach that has been used is to compare the calculated volume of connection voids to the grout volume used. However, for the potentially small grout volume likely to be used in a precast connection, this may be difficult, especially since grout that exits vent tubes must also be carefully retained and measured. Even if an engineer were able to predict from calculations and measurements that some of the connection has not been grouted, it would be very difficult to quantify the impact of relatively small embedded air voids on connection performance. For this reason, it is critical that special attention be paid to quality control prior to and during grouting operations.

2.4.2.2 Pressure Grouting

Pressure grouting involves pumping grout into connections under pressure. As portrayed in Figures 2.10 and 2.12, grouted sleeve couplers use a bottom inlet port and a top outlet port to ensure full removal of air. This approach may similarly be applied to the other connection types, such as grouted vertical ducts. After sealing the bedding layer, grout is pumped through one inlet port to displace the air. Vent tubes are plugged as grout flows out, causing the ducts to fill upward. The duct nearest to the inlet port will fill first, and depending on the grout viscosity, other ducts may need to be topped off. After topping off the ducts, the inlet port is plugged. As shown in Figure 2.29, rectangular and square columns (or piles) should use four ports. Round columns may use two to four ports.

2.4.2.3 Quality Control

Quality control is critical to grouting operations. The following is a list of quality control aspects that require special attention:

1. connector size and length
2. connection dimensions, such as duct diameter and depth
3. connection surface and bar cleanliness
4. grout type and quantity
5. grout mixing procedures
6. grout pressurizing procedures
7. bedding formwork
8. grouting of the bedding layer
9. complete grouting of the connection
10. curing
11. protection against disturbing bars during curing
12. prevention of embedded air

Check lists such as that shown above are recommended for predefined stages of construction, such as placement of corrugated ducts in the cap and connectors in a column or pile. In addition, grout manufacturers' recommendations should be followed precisely. Additional issues are listed in subsequent chapters, based on Phase 1-3 tests.

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CHAPTER 3:
PHASE 1 PULLOUT TESTS

3.1 INTRODUCTION

The development of connection details in Chapter 2 revealed numerous uncertainties associated with transfer of connection forces. Of the four connection alternatives, grout pocket and grouted vertical duct connections presented the most uncertainties, such as bar anchorage within grout, interlock of pockets and ducts within a precast cap, failure modes, and the influence of confining reinforcement. Phase 1 testing was designed to investigate these issues through bar pullout tests.

Pullout tests provide a number of important advantages. Connectors in an actual precast bent cap system are subjected to a complicated state of stress associated with axial forces, shears and moments. Through Phase 1 pullout tests like that shown in Figure 3.1, connectors were subjected to the simpler condition of axial tension alone. This facilitated an investigation of the fundamental bond and load-deflection characteristics of grouted connections. A clear understanding of this behavior provided a basis for developing Phase 2 connection details and for understanding the more complicated connection behavior examined in Phase 2 testing. Finally, because pullout tests involve relatively simple test specimens and test configurations, a large number of variables may be investigated.

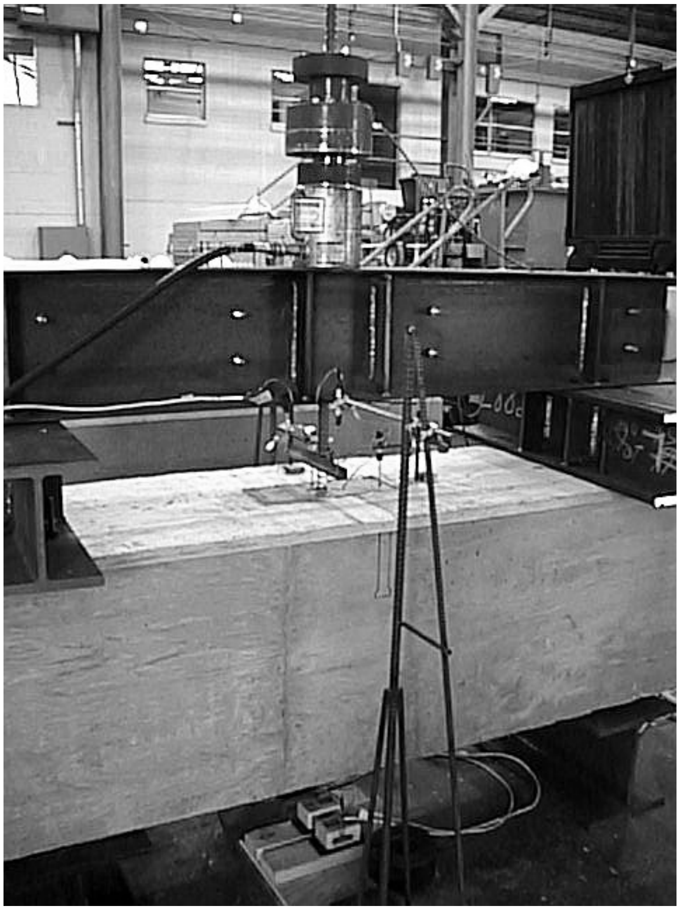


Figure 3.1 Setup for Phase 1 Pullout Test

This chapter summarizes the Phase 1 pullout test program for single-line and double-line grout pockets, as well as grouted vertical ducts. First, a synopsis of the test program is provided, including a discussion of the test variables, scope of testing, casting and grouting of specimens, instrumentation, test setup, and test procedure. Then, test results are discussed for each connection type. An additional discussion of Phase 1 tests is presented in Reference 3.1.

3.2 SYNOPSIS OF TEST PROGRAM

3.2.1 Variables

Variables investigated for grout pocket and grouted vertical duct connections included:

- Bar anchorage—upset-headed and straight epoxy-coated rebars
- Bar size—#6, #8, and #11 rebars
- Embedment depth—5 to 18 times the bar diameter (d_b)
- Bars per pocket—single and double bars
- Bar configuration—transverse moment and longitudinal moment
- Confining reinforcement—spiral and welded wire fabric
- Grout type—neat and extended grout
- Grout brand—Masterflow 928, Euclid High Flow, and Sika 212

The following sections provide rationale for the use of these test variables.

3.2.1.1 Bar Anchorage

Upset-headed bars were selected for initial tests because they provide a balance in satisfying opposing requirements for bar anchorage and tolerances. The smaller upset heads more readily accommodate horizontal tolerances than other headed bars, yet also provide better anchorage than straight bars (Figure 3.2). For comparison, straight #8 and #11 bars were also tested. Both epoxy-coated and black bars were originally planned for testing. However, black bars were later eliminated from the test matrix when epoxy-coated bars exhibited adequate bond.

3.2.1.2 Bar Size

Three bar sizes were used in Phase 1 testing: #6, #8, and #11 bars. Because of the recognized difficulty in scaling bond behavior, the larger #8 and #11 bars shown in Figure 3.3 were used as connectors in the majority of the test specimens, which were sized to represent approximately full-scale connections for single-line grout pockets and grouted vertical ducts. In addition, a limited number of tests representing ¾-scale connections were also conducted. The #6 connectors approximately represented #8 bars at ¾-scale, as well as a lower bound size for connectors in trestle pile bents. The use of the smaller #6 bars also enabled the influence of bar diameter on bond to be examined over a broader range of bar sizes. By selecting these three connector sizes, all Phase 1 specimens were able to use the same formwork, significantly expediting testing.

3.2.1.3 Embedment Depth

The failure mode of a connector subjected to tension is dependent on the depth the connector is embedded in the connection. Embedment depth for headed bars is the length of the bar, without the head, embedded in the connection. For straight bars it is merely the embedded length of the bar. For headed anchors in concrete without edge effects, the failure mode normally shifts from concrete breakout to bar yield as the embedment depth increases. The latter failure mode is more ductile and thus preferable.

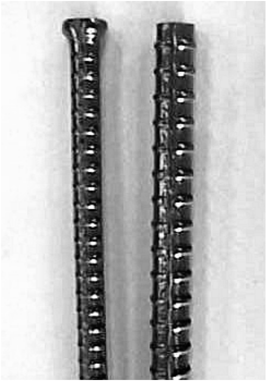


Figure 3.2 Headed and Straight Epoxy-coated Bars used in Phase 1 Tests

Figure 3.3 Two Bar Sizes used in Phase 1 Tests

Varying the embedment depth in Phase 1 tests was considered crucial to understanding connection behavior, failure modes, and the relationship between anchorage in grout pockets and concrete. Because behavior was not well understood prior to testing, behavior of the earlier tests provided guidance in selecting embedment depths for later tests to ensure different failure modes were produced. Embedment depths ranged from 5 to 18 times the bar diameter ($5d_b$-$18d_b$).

3.2.1.4 Number of Bars

To help establish behavioral characteristics, initial tests were conducted with a single bar grouted in a single-line grout pocket (Figure 3.1). However, in practice single-line and double-line grout pockets will use two or more connectors. Therefore, after studying the pullout behavior of single bars grouted in a pocket, multiple-bar behavior was investigated by grouting and testing two bars in a pocket (Figure 3.6).

3.2.1.5 Bar Arrangement

Actual precast bent cap connections will be subjected simultaneously to both transverse and longitudinal moments. Figure 3.4a shows how two connectors can be placed in tension in a double-line grout pocket to represent the effects of a longitudinal moment. In contrast, Figure 3.4b shows one connector in tension for each pocket, representing the effects of transverse moment. Tests were conducted to directly compare response for these cases. For simplicity in studying behavior and in conducting tests, the effects of simultaneous moments were neglected. However, Phase 2 testing addresses this loading condition.

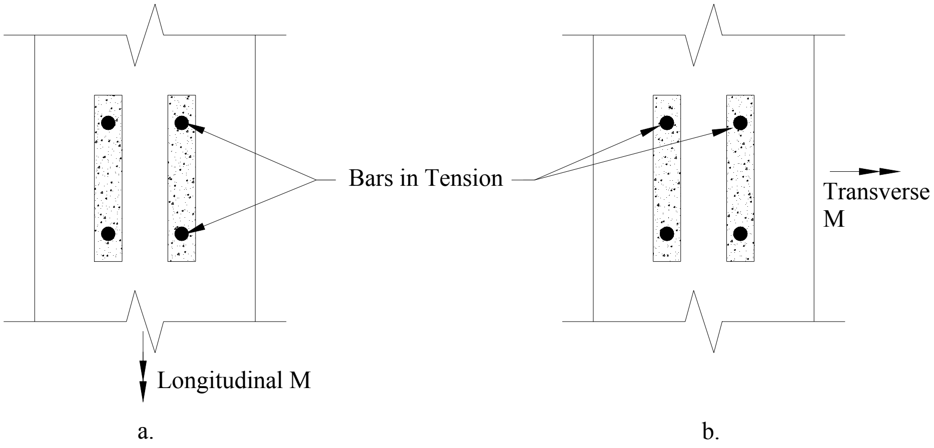


Figure 3.4 Longitudinal vs. Transverse Moments in Double-Line Grout Pockets [3.1]

3.2.1.6 Confining Reinforcement

Confining reinforcement was expected to provide increased ductility, greater strength, and possibly smaller service-level crack widths. To investigate the influence of confining reinforcement on grout pocket connections, tests were conducted both with and without confining reinforcement. Confining reinforcement was not used around grouted vertical duct connections, although the steel ducts and concrete were expected to provide confinement to connectors.

Grout pockets were confined with either spiral reinforcement or welded wire fabric. Spiral reinforcement is inexpensive, readily available, and easy to place, although for typical pocket shapes spiral confinement will not closely surround all sides of a pocket. In contrast, confinement consisting of welded wire fabric may be formed to match the shape of grout pockets more closely, but requires a greater effort in fabrication and uses a shape susceptible to greater deformation before development of significant confining forces. Welded wire fabric was also used in testing to ensure that steel would cross the horizontal cracks observed in some tests.

3.2.1.7 Grout Extension

Based on the results of trial batches and grout testing mentioned in Chapter 2, considerable uncertainty existed regarding the use of extended grout for precast bent cap systems. Initial tests compared the performance of neat and extended grouts to address issues of flowability, compatibility, and strength. Masterflow 928 grout was compared to Masterflow 928 grout extended with 3/8-in. pea gravel.

3.2.1.8 Grout Brand

Trial batches demonstrated that many properties differ among proprietary grouts. To examine potential differences in connection behavior, pullout tests were conducted using Masterflow 928 (MF928), Euclid High Flow (EHF), and Sika 212. To limit the scope of Phase 1 testing, MF928 was used for most tests, with EHF and Sika 212 used for four comparison tests. Appendix A provides grout product data sheets.

3.2.2 Specimen Fabrication

Pullout tests were conducted using bars grouted into pockets or ducts that were cast into beam specimens. Two specimen sizes were used, one for single-line and double-line grout pocket tests, and another for grouted vertical ducts.

3.2.2.1 Single-Line and Double-Line Grout Pocket Specimens

Figures 3.5 and 3.6 show plan, elevation, and section views for single-line and double-line grout pocket specimens, respectively. Specimens were 2 ft. x 2 ft. x 12 ft. beams, representing full-scale connections for single-line grout pockets and ¾-scale connections for double-line grout pockets. Note that pockets were inverted to accommodate a simpler upward loading of the bars in the test setup (Figure 3.1). The specimen cross section was selected to represent realistic rectangular bent cap dimensions, whereas the length was chosen to permit two tests to be conducted, one on each end, without interference. Cap reinforcement was necessary to resist the applied loads, as well as to include any effects on connection behavior. Longitudinal reinforcement ratios were based on the Red Fish Bay precast caps. However, Red Fish Bay stirrups were eliminated in the connection region to simplify the connection detail and to prevent any contribution to failure capacity. Figures 3.7 and 3.8 respectively show schematics of the rebar cages used for single-line and double-line grout pocket specimens. Figure 3.9 shows the corresponding formwork.

Most grout pockets were formed using plywood inserts sheathed in metal flashing (Figure 3.10). Flashing facilitated form removal after casting and allowed investigation of a smooth surface at the concrete-grout interface. Single-line grout pocket inserts incorporated a single 5-degree taper through the depth of the cross section to enhance anchorage of the pocket within the cap. A small 1-degree taper was included along the length of the cap to facilitate form removal. In contrast, double-line grout pockets were nearly vertical through the cross section to allow for two pockets, but incorporated a 7-degree taper along the length for anchorage. For stability during casting, inserts were anchored to the bottom formwork, and tie-downs were attached to the tops of inserts, as shown in Figure 3.7.

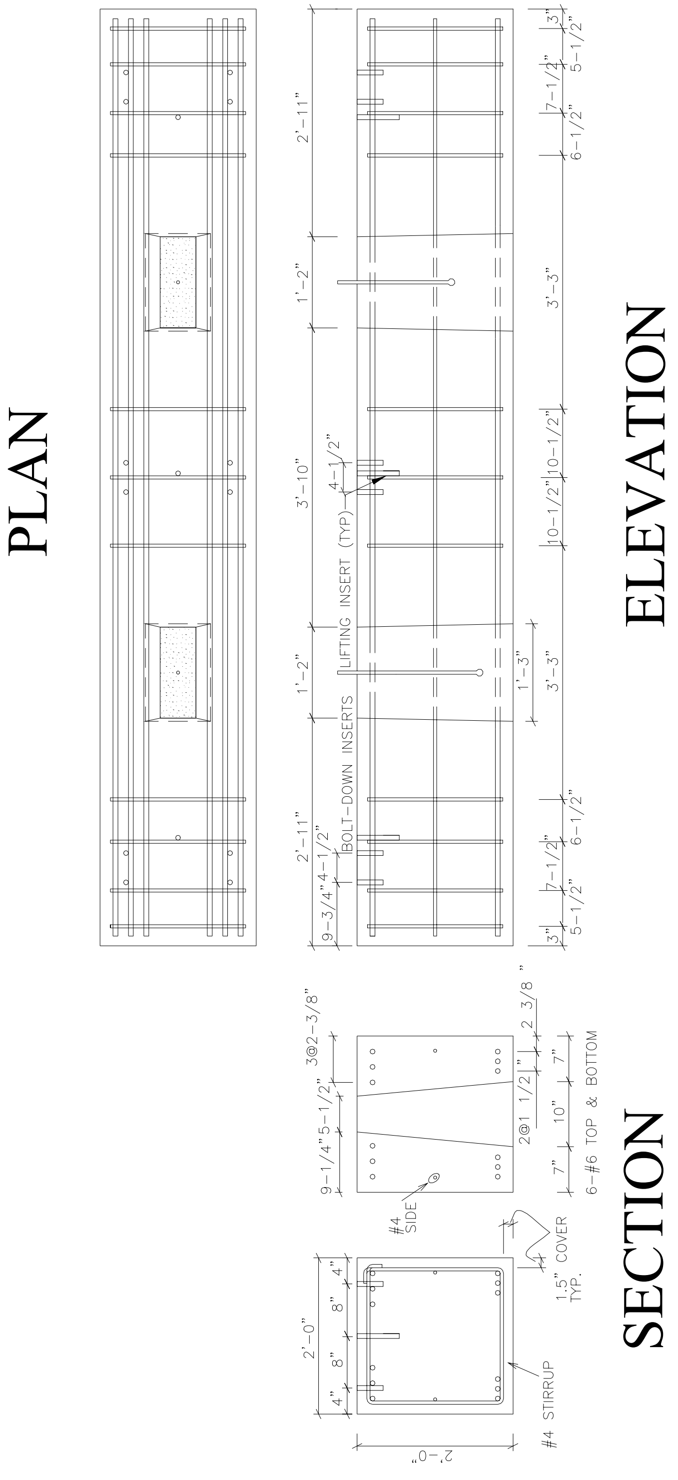


Figure 3.5 Single-Line Grout Pocket Specimen

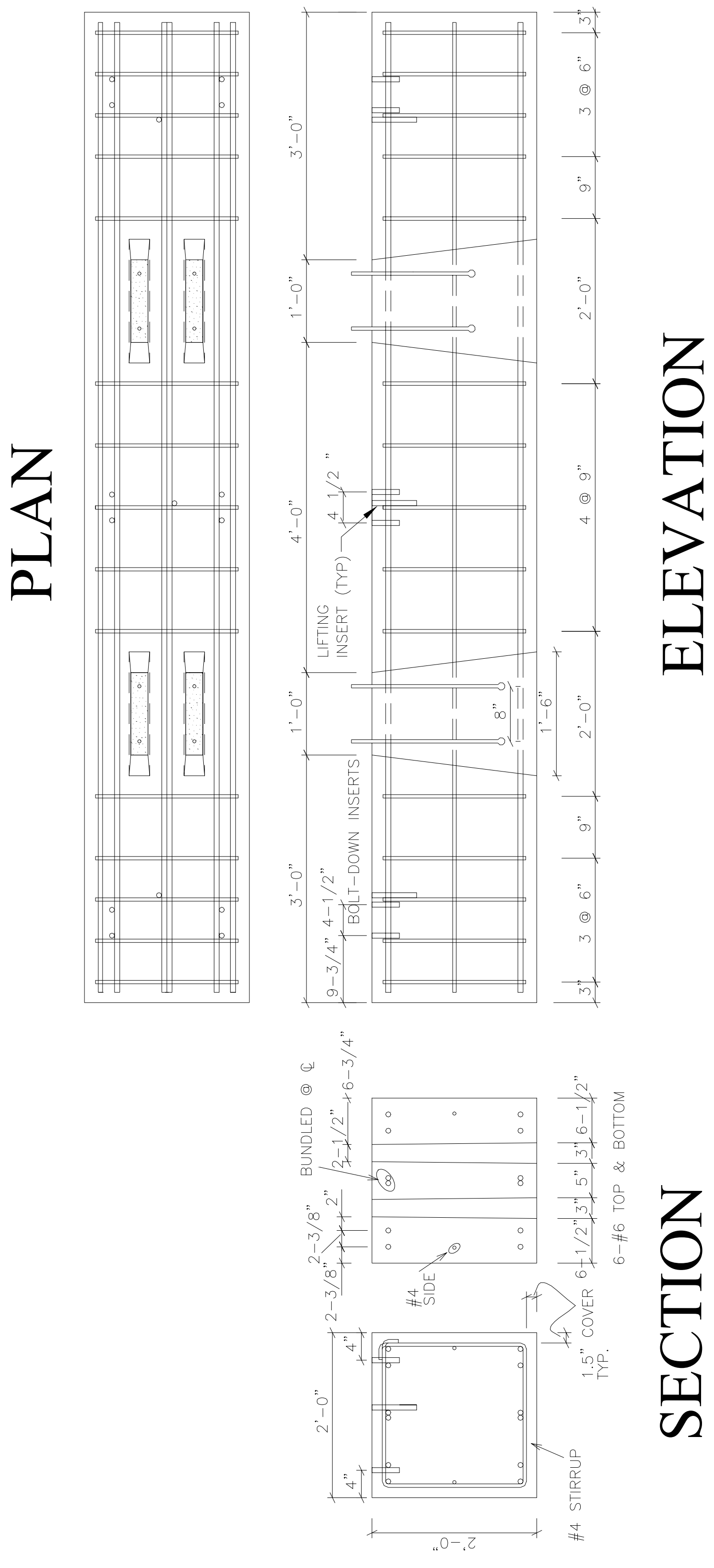


Figure 3.6 Double-Line Grout Pocket Specimen

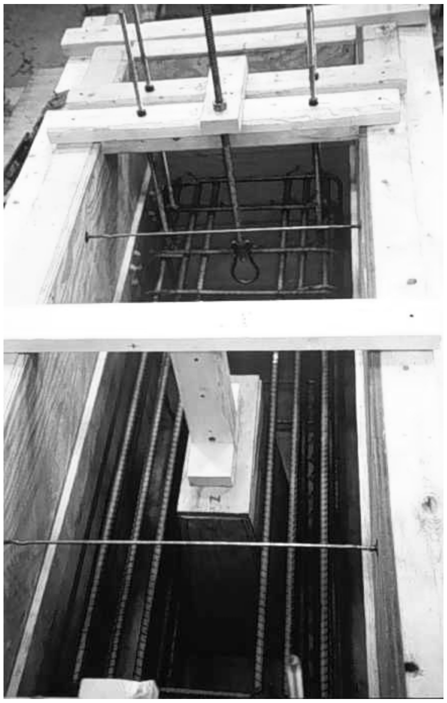


Figure 3.7 Single-Line Rebar Cage

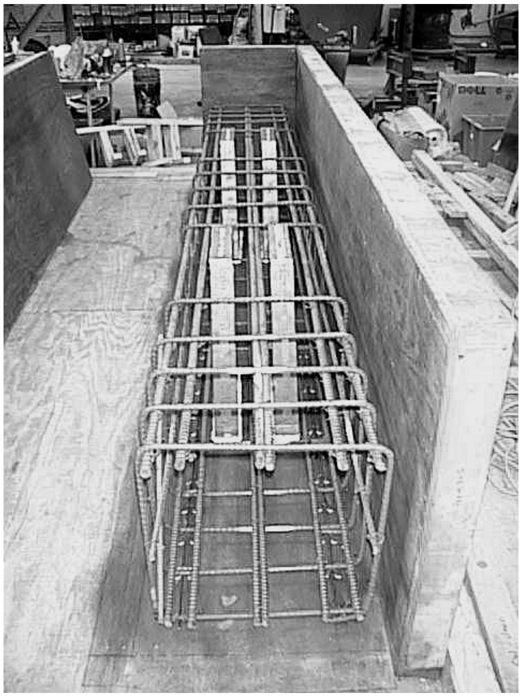


Figure 3.8 Double-Line Rebar Cage

Figure 3.9 Specimen Formwork

Figure 3.10 Single-Line Grout Pocket Insert

Confining reinforcement was used in single-line and double-line grout pocket tests (multiple-bar cases). Two confinement options were investigated: spiral reinforcement and welded wire fabric. Figures 3.11 and 3.12 show these options. Because of the low cost and relative ease of placement for spiral reinforcement, a tight 2.5-in. pitch was used with #3 spirals. Auxiliary vertical bars were used to maintain the pitch and were positioned to minimize any contribution to strength. A 4 x 4 x 6 gage welded wire fabric mesh was also selected for confinement. Figure 3.13 demonstrates the greater fabrication effort required for forming the mesh to shape. Confining reinforcement was limited to a height of 18 in. to fit between the top and bottom layers of longitudinal reinforcement (Figure 3.11). Table 3.1 summarizes the dimensions and percentage of confining reinforcement. The welded wire fabric mesh has a confinement ratio (based on horizontal wires) of approximately 20% of that of the spirals.

Table 3.1 Phase 1 Confining Reinforcement

Connection Type	Confinement	Description	Volumetric %
Single-Line Grout Pocket	Spiral	#3 @ 2.5 in. 18 in. diameter	0.96
	Welded Wire Fabric	4 x 4 x 6 gage 17-¼ x 12-¼ x 18 in.	0.20
Double-Line Grout Pocket	Spiral	#3 @ 2.5 in. 20-½ in. diameter	0.86
	Welded Wire Fabric	4 x 4 x 6 gage 15-½ x 22 x 18 in.	0.16

Concrete was cast in two lifts using a hopper supported by an overhead crane (Figure 3.14). Each lift was vibrated and the surface finished with trowels. Forms were covered with plastic until the concrete set (four hours), after which wet burlap was placed for three days of moist curing. Forms were typically removed three to four days after casting. After stripping the forms, the specimens were set on support blocks. Due to shrinkage of the concrete, a sledgehammer was needed to dislodge the pocket forms from the specimens. A single-line grout pocket specimen is shown in Figure 3.15 after the inserts were removed.

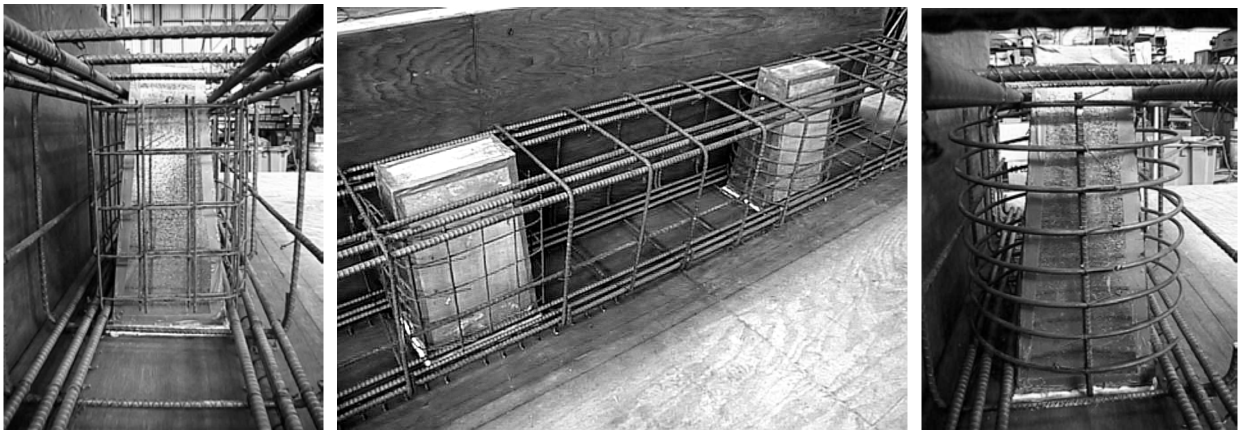


Figure 3.11 Confinement used for Single-Line Grout Pockets

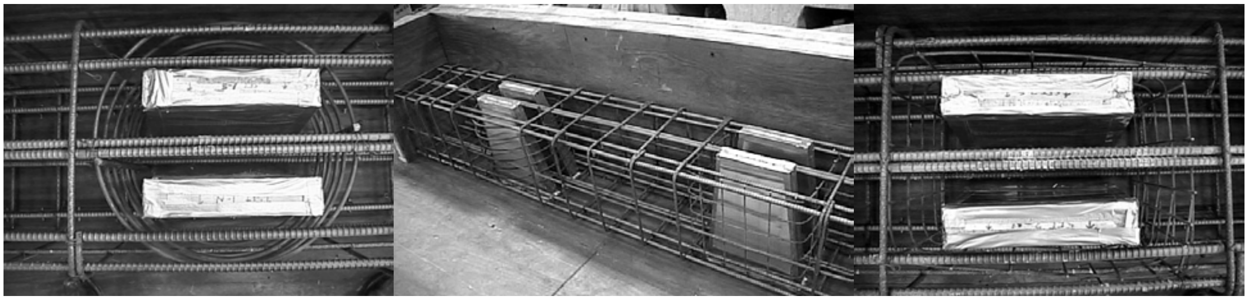


Figure 3.12 Confinement used for Double-Line Grout Pockets

Figure 3.13 Preparation of Welded Wire Fabric Confinement

Figure 3.14 Specimen Casting

Figure 3.15 Specimen after Removal of Grout Pocket Inserts

3.2.2.2 Grouted Vertical Duct Specimens

Figure 3.16 shows plan, elevation, and section views for grouted vertical duct specimens. Specimens were 30 in. x 24 in. x 12 ft. beams, which were 6 in. deeper than single-line and double-line specimens. This increased depth served three functions: 1) to accommodate deeper embedment depths required for #11 bars; 2) to provide additional resistance for the larger pullout forces; and 3) to enable the same formwork to be used for all Phase 1 specimens. Four-inch diameter, semi-rigid, spirally-crimped (corrugated) steel ducts were used for each test. Ducts extended through the entire beam depth. Because of the expected localized failure, four ducts were cast in each specimen, as shown in Figure 3.17.

Only slight modifications to the rebar cage were necessary to accommodate corrugated ducts. The plan view in Figure 3.17 shows the extra rebars used to stabilize the ducts at the top and bottom of the rebar cage. Ducts were sealed at both ends to prevent concrete from entering the ducts during casting. After form removal, some ducts were scrubbed with a wire brush to help remove a thin layer of paste that formed on the inside surface due to seepage of moisture through the ducts. Following this, the specimens were ready for grouting.

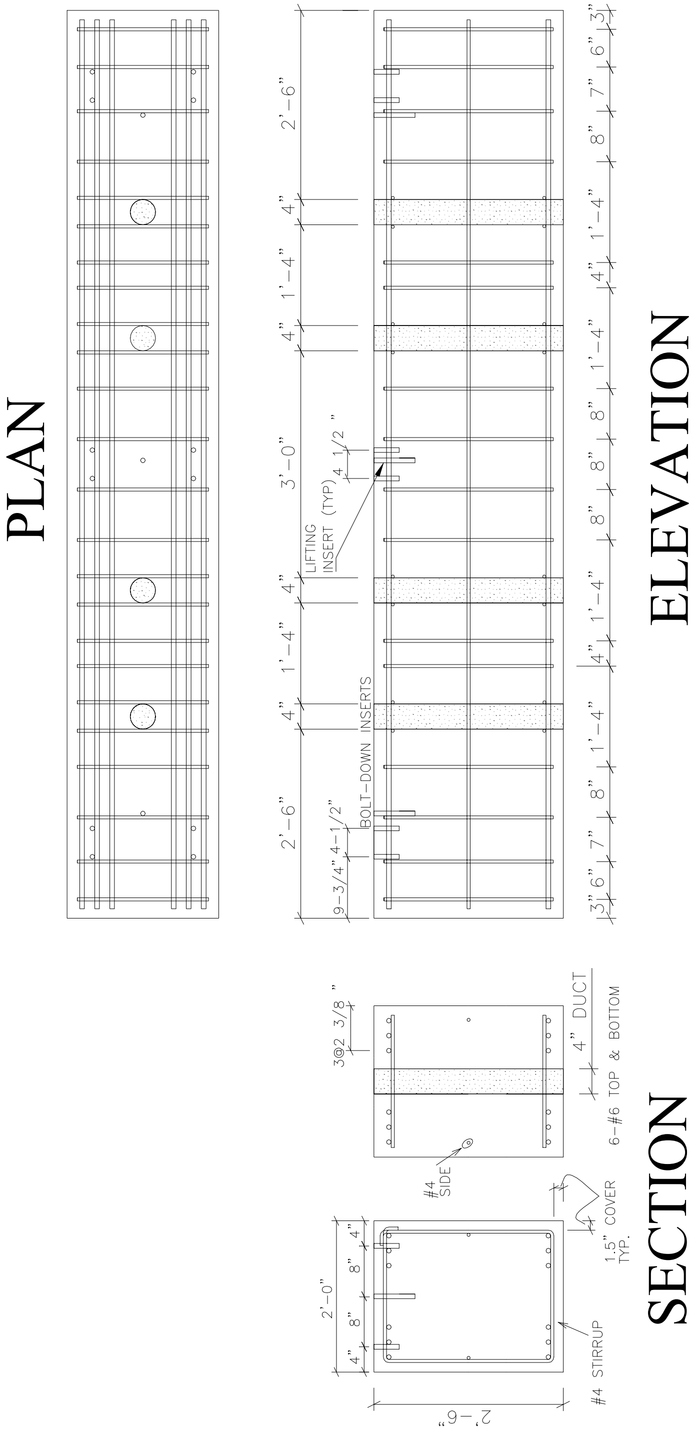


Figure 3.16 Grouted Vertical Duct Specimen

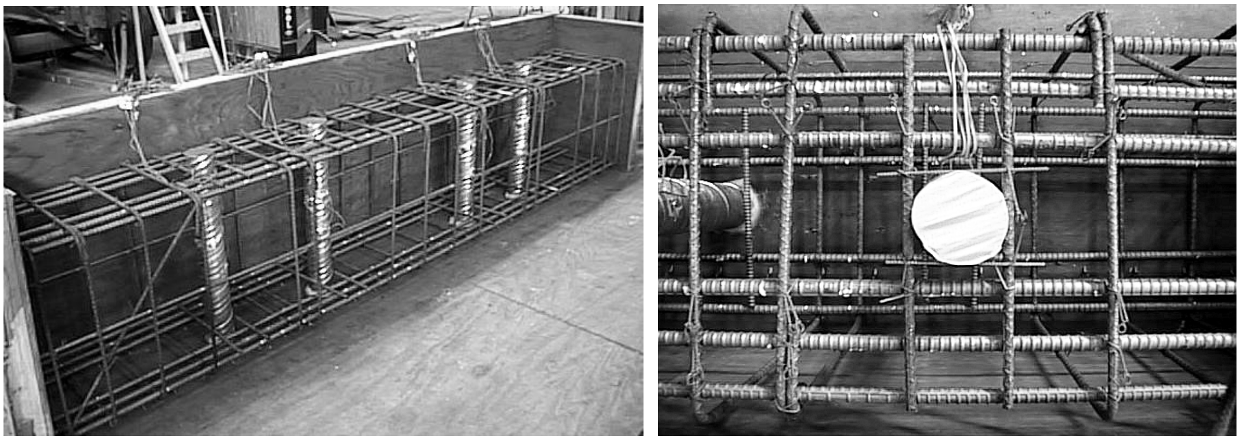


Figure 3.17 Grouted Vertical Duct Rebar Cage and Ducts

3.2.3 Grouting

3.2.3.1 Grouting Operations

Following casting, grout pocket specimens were gravity-flow grouted in the following manner:

1. The bottom opening of the grout pocket was formed with plywood and sealed with caulk.
2. The pocket was then filled with water to presoak the pocket surface in order to minimize the loss of water to the surrounding concrete (Figure 3.18). Although manufacturers typically suggest presoaking for 24 hours, presoaking was limited to approximately one to three hours to simulate the more likely pre-watering that will take place in the field (based on comments from the Industry Review Committee).
3. The pocket was vacuumed empty about one hour prior to grouting.
4. Connectors with strain gage instrumentation were placed in the pocket, as shown in Figure 3.19 and then plumbed. A threaded rod and copper sheathing extended from the head of the bar through the bottom formwork (for displacement measurements), and lead wires extended from strain gages out the top of the pocket. Figure 3.20 shows the different connector configurations used in multiple-bar tests.
5. Grout was mixed in a 3.5-ft^3 paddle-type mortar mixer (Figure 3.21). Manufacturer's recommendations were followed for mixing. This included using the grout within the recommended temperature range, adding mixing water to the mixer before the packaged grout, and mixing the grout for the recommended time. Initially, 10 lbs of water was added per bag (50-55 lbs). Grout was typically mixed for 3-5 min.
6. After mixing, the neat grout was tested using the ASTM C 939 Flow Cone test (Figure 2.24). Grout with a flow between 10 and 30 sec (fluid classification per ASTM C 939) was normally used. However, to avoid remixing, efflux times as large as 46 sec were accepted. If the flow was larger than 46 sec, an additional 10% water was added and the grout remixed. Remixing usually reduced the flow to less than 30 sec. Table 3.2 summarizes flow cone results for most tests. Extended mixes were not tested for fluidity, although the flow was sufficient for filling single-line grout pocket specimens.
7. Pockets were grouted with a funnel and a 1.25-in. (inside) diameter tube, as shown in Figure 3.22. Two methods were used in filling the grout pocket: 1) tremie tube, and 2) modified tremie tube. Initially the tremie tube approach was used, but resulted in air passing through the funnel and into

the grout, as a constant grout flow was not maintained throughout the entire grouting operation (i.e., when refilling buckets). To prevent this, a modified approach was used, in which the tube was positioned against the side face just above the level of grout, rather than being submerged in the grout. In some cases, grout was also lightly tamped with a rod to help remove air.

8. Curing compound was placed on the surface within 30 min. after grouting to prevent loss of moisture. Manufacturers recommended that wet rags be placed on exposed surfaces immediately after grouting. However, this was not done to preserve a smooth surface for testing. After the top surface hardened (3 to 4 hours), wet rags were placed to moist cure the grout for 24 hours. The bottom form was typically removed after 1 to 3 days. Figure 3.23 shows a pocket after completion of grouting.

Vertical ducts were grouted in the same manner, except without prewatering. A smaller tremie tube (5/8-in. inside diameter) was required for grouting the 4-in. ducts.

Figure 3.18 Presoaking of Grout Pocket

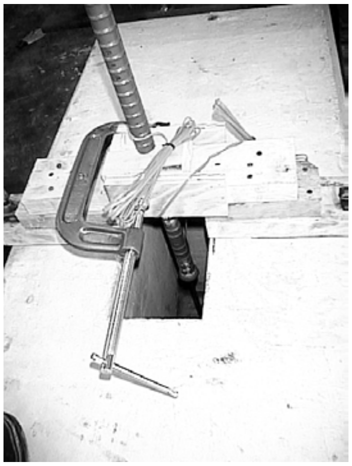
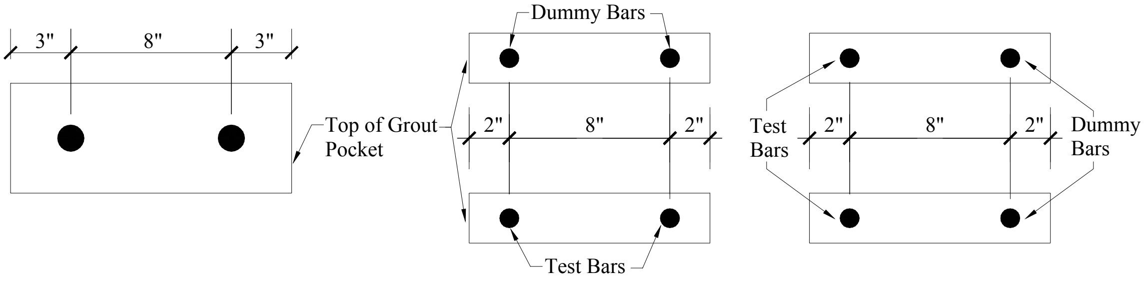


Figure 3.19 Support of Connector before Grouting



SLGP

DLGP
Longitudinal M

DLGP
Transverse M

Figure 3.20 Bar Configurations for Multiple Bar Tests [3.1]

Figure 3.21 Grout Mixing

Figure 3.22 Gravity-flow Grouting

Figure 3.23 Single-Line Grout Pocket after Curing

3.2.3.2 Air Voids

Air voids appeared at the surface of many grouted specimens. A number of factors contributed to the formation of air voids during gravity-flow grouting, including, high grout viscosity, grout remixing, grout handling, and the grouting technique. When the grout viscosity was too high, entrapped air could not quickly rise to the surface before the grout stiffened. In this case, entrapped air surfaced so slowly that voids formed at the surface, often adjacent to a connector. Other factors, such as remixing, handling, and grouting methods, increased the likelihood of air voids by entrapping additional air into the grout during grouting operations. Insufficient presoaking might also contribute to air embedment for grout pocket connections.

Depending on the grout mix, bubbles and voids appeared differently at the surface (compare Figures 3.24a-3.24d). MF928 and Sika 212 exhibited relatively few bubbles and voids, whereas EHF manifested a very large number of small bubbles, which resulted in a pumice-like texture over part of the surface. Venting occurred over a period of approximately 1 to 2 hours. In several cases, red dye was injected into air voids with an eyedropper to determine the depth of penetration. Continuous air paths typically penetrated no more than one to two inches beneath the surface (Figure 3.25).

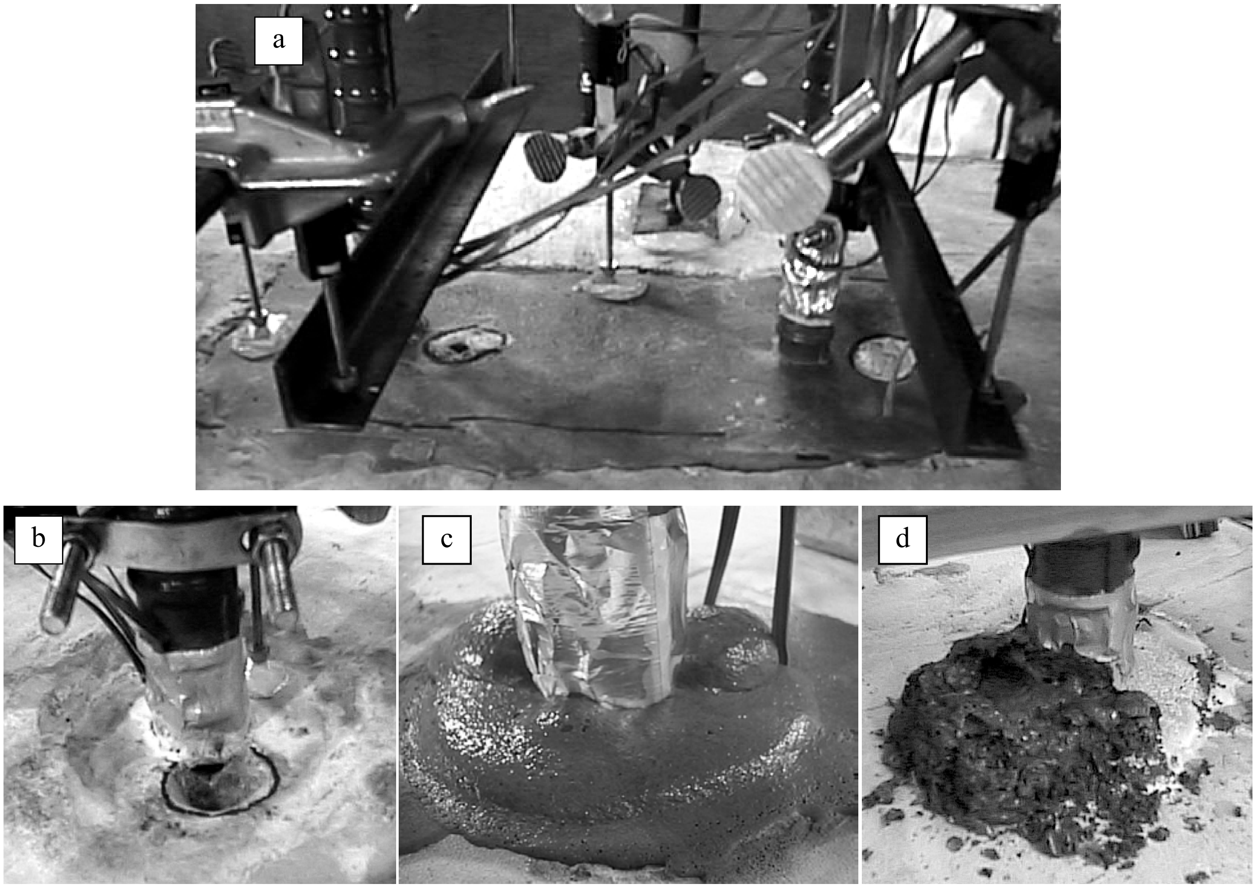


Figure 3.24 Bubbling and Air Voids—a) MF928, b) Sika 212, c) MF928, d) EHF

Figure 3.25 Dye Injected into Air Voids

3.2.3.2.1 Grout Viscosity and Grout Remixing

Phase 1 grouting demonstrated that a sufficiently low grout viscosity is an important factor in preventing air voids. During grouting operations, an efflux time of 30 seconds or less was targeted. In addition, remixing was avoided as much as possible. Although remixing usually reduces flow times, it may cause a reduction in strength and durability properties, and add air into the mix. As a compromise, some grout mixes with slightly larger flows were accepted without remixing, after early tests demonstrated that small air voids had no discernable influence on pullout stiffness or strength.

Data in Table 3.2 demonstrate that there is a correlation between flow time, remixing, and surface voids. Three specimens listed in the table exhibited no surface voids: DL05, DL06, and SL07. DL05 and DL06 achieved flows of 21 and 28 sec, respectively, and were prepared at a temperature of 70° F and without remixing. SL07 was remixed twice and portrayed an unusually stiff consistency a short time after the flow cone test. Although this prevented surface voids and did not affect strength, it unduly complicated grouting operations.

A relatively small number of air voids (one to two) appeared at the surface for five of the six batches that achieved a flow less than 30 sec without remixing (DL01, DL04, VD01-VD03, VD06-VD07). Each of these pockets was grouted with MF928. The sixth batch, VD06, used Sika 212 grout and had the lowest recorded flow (13 sec), but also exhibited excessive bleeding through a very large void adjacent to the bar (Figure 3.24b). VD06 demonstrates that a low efflux time is a necessary but insufficient condition for acceptable grout performance.

Grouts that were remixed (SL12, SL17, DL02, VD04, VD08) achieved a final flow of 34 sec or less. However, a larger number of air voids formed at the surfaces, compared to cases in which the grout was not remixed. An acceptable flow was achieved at the expense of additional air embedment and surface voids. A comparison of the grouted surfaces for DL01 and DL02 confirms this, as more voids appeared for DL02, which was remixed, than for DL01, which was not. For this reason, trial batches should be conducted on site to ensure a suitable grout flow will be achieved without remixing.

All three tests that used EHF grout required remixing. It is believed that the porous texture observed at the surfaces for these cases (e.g., Figure 3.24d) was the result of expansion mechanisms in the Grade C grout. Air voids were also present. The impact of this on a precast bent cap connection was investigated further in Phase 2.

3.2.3.2.2 Grout Handling and Grouting Operations

Grout handling and grout operations also influenced air voids. It was observed that pouring grout into the buckets from the mixer could entrap an additional amount of air. To minimize this effect, the bucket was held close to the mixer when pouring. In addition, it was found during the first two tests that air was added to the grout when a continuous flow was not maintained in the funnel. Because of this, a modified tremie tube approach was adopted. Phases 2 grouting used the tremie tube approach with a constant flow.

Table 3.2 Flow Cone Efflux Times and Surface Air Void

Test	Grout Brand	Temp (F)	Flow (sec)	Remix	Final Flow (sec)	Air Voids at Surface
SL05	MF928	85	40	N/A	N/A	Several large voids
SL07	MF928	93	40	20% water + remix1 30% water + remix2	N/A 25	None
SL11	MF928	75	35	N/A	N/A	One large void
SL12	EHF	75	45	W/o added water	< 45	Many small bubbles (related to mix)
SL15	MF928	67	46	N/A	N/A	Several large voids
SL16	CIP	N/A	N/A	N/A	N/A	Several large voids
SL17	MF928	77	60	10% water	20	Several large voids
SL18	MF928	77	15	N/A	N/A	Few small voids
DL01	MF928	70	24	10% water before initial mixing	N/A	Fewer voids than DL02
DL02	MF928	70	45	10% water	17	Several large voids
DL03	MF928	72	39	N/A	N/A	Several small voids
DL04	MF928	69	21	N/A	N/A	Several small voids
DL05	MF928	70	21	N/A	N/A	None
DL06	MF928	70	28	N/A	N/A	None
VD01-02	MF928	72	20	N/A	N/A	Small voids
VD03, 06-07	MF928	72	25	N/A	N/A	Small voids
VD04	EHF	68	110	10% water	33	Large amount of bubbles/air voids
VD05	Sika 212	72	13	N/A	N/A	Large void; alot of bleed water
VD08	EHF	72	120	10% water	34	Large amount of bubbles/air voids

3.2.3.3 Lessons Learned

Several lessons were learned through Phase 1 grouting. Although grout pockets were inverted and some operations did not match actual field grouting operations, a number of useful guidelines were established for application to a precast bent cap system:

Grout fluidity should be determined in accordance with the CRD-C 611/ASTM C 939 Flow Cone Method. A flow of 15 to 30 sec should be required. Temperature requirements for proprietary grouts should be carefully observed. For some grout brands, this may necessitate the use of special cold or hot weather practices in Texas for much of the year.

Trial batches of grout should be required and mixed to determine the amount of water required to achieve an acceptable flow for actual field conditions. The flow at the estimated time of completion for grouting operations should also be determined. Remixing should not be permitted. Trial batches should also be examined for segregation and bleeding. Grouts that exhibit undue segregation or bleeding should not be used.

Sufficiently long grout working times should be provided. An accurate estimate of the required time for grouting operations should be determined to ensure connections are grouted before a significant loss of grout fluidity.

Precautions should be taken to minimize air embedment when pouring grout from the mixer to dispensers and when grouting the connection. In addition, grout pocket connections should be presoaked a minimum of two hours to prevent potential air entrapment as well as loss of moisture from the mix.

When using a tremie tube, grout operations should be practiced to ensure that the method used (continuous or modified tremie tube) will prevent air entrapment and that equipment such as tube length, tube diameter (inside and outside), and funnel volume are adequate for actual grouting.

3.2.4 Materials

3.2.4.1 Steel

ASTM A615 Grade 60 reinforcing steel was used for longitudinal rebars, stirrups, and spiral confinement in the beam specimens, as well as for connectors. ASTM A497 Grade 70 deformed welded wire fabric was used for mesh confinement. Headed Reinforcement Corporation (HRC) in Fountain Valley, CA used their *Upsetter* to forge upset heads on the ends of straight bars (Reference 3.2). Fletcher Coatings Company of Orange, CA coated headed and straight test bars with 3M Scotchkote 426 epoxy coating (Figure 3.3; Reference 3.3). Yield and fracture strengths for connectors are listed in Table 3.3. The higher mill report strengths are attributed to the higher loading rate used in mill tests than in laboratory tests of rebar and specimens. Strengths of bars used in pullout tests are expected to match laboratory test results. Corrugated duct material was galvanized, cold-rolled steel per ASTM A619 and ASTM A527. Ducts had a corrugation height of 0.094 in. and a 26-gage (0.23 in.) thickness.

Table 3.3 Steel Connector Strengths (ksi)

	Mill Report		UT Laboratory	
Bar Size	**Yield**	**Fracture**	**Yield**	**Fracture**
#6	72	101	65	94
#8	74	101	69	97
#11	70	100	59	85

3.2.4.2 Concrete

Concrete used for specimens was the standard TxDOT Class C mix, with a minimum 28-day compressive strength of 3600 psi. Table 3.4 summarizes the mix design. The use of 3/4-in. maximum coarse aggregate and a slump of approximately 4 in. facilitated consolidation of the concrete in the cage and around grout pocket inserts. ASTM standard 6 x 12 in. cylinders cast with each batch of concrete were tested at 3, 7, 14, 28, and 56 days, as well as on test days. Compressive strength curves are plotted in Figure 3.26 and show an average 28-day strength of approximately 5400 psi.

Table 3.4 TxDOT Class C Concrete Mix Design

Cement (lbs/cy)	¾-in. Coarse Agg. (lbs/cy)	Fine Agg. (lbs/cy)	Water (lbs/cy)	Retarder (oz.) (lbs/cy)
564	1882	1191	250	24

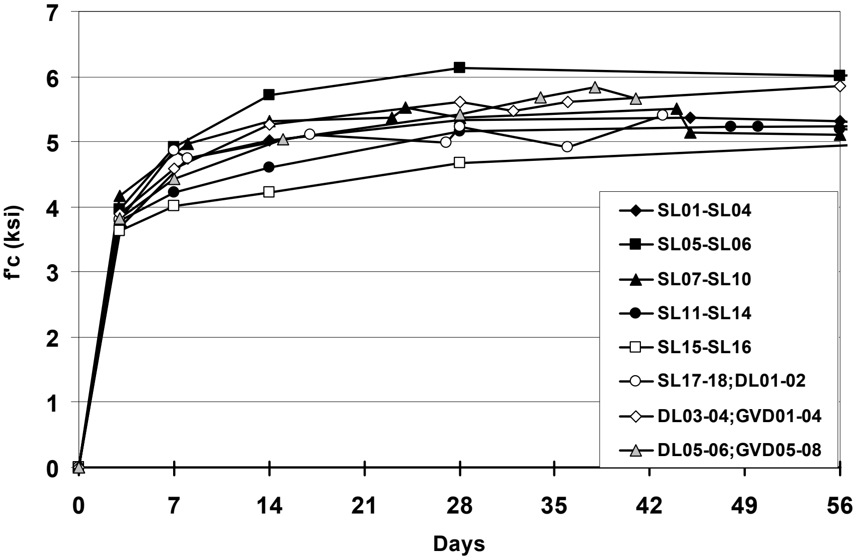


Figure 3.26 Concrete Compressive Strength Curves

3.2.4.3 Grout

Grouts manufactured by Master Builders Technologies, Euclid Chemical Company, and Sika Corporation were described in Chapter 2. Based on preliminary tests and further telephone conversations, the following non-shrink grouts were used for Phase 1 testing: 1) Masterflow 928, 2) Euclid Hi-Flow, and 3) Sika 212. Euclid Hi-flow grout was used in lieu of Euclid NS (Non-shrink) grout, according to the manufacturer's recommendation. Master Builders Technologies and Euclid Chemical Company donated grout for testing.

As mentioned earlier, Masterflow 928 was used both with and without extension for comparison. Extended grouts used 25 lbs of 3/8-in. river gravel per 55-lb bag. Table 3.5 shows a sample gradation for the coarse aggregate. All grouts were mixed according to manufacturer's recommendations as described in Section 3.2.4.1.

Table 3.5 Sample Gradation of Pea Gravel Used in Extended Grout

Sieve	% Retained on Sieve
3/8-in.	4
¼-in.	63
#4	21
#8	9
<#8	3

Restrained 2-in. cubes were prepared according to ASTM C 109 for each batch of grout (Figure 3.27). In most cases, cubes were tested at 1, 3, 7, and 28 days, as well as on test day. Four-inch cylinders were used for extended grout mixes. Figure 3.28 compares grout strength for the different mixes. A 0.8 conversion factor was applied to cube strengths. Grout strength was expected to exceed concrete strength on test day, although this did not always occur.

Figure 3.27 Preparation of Grout Cubes

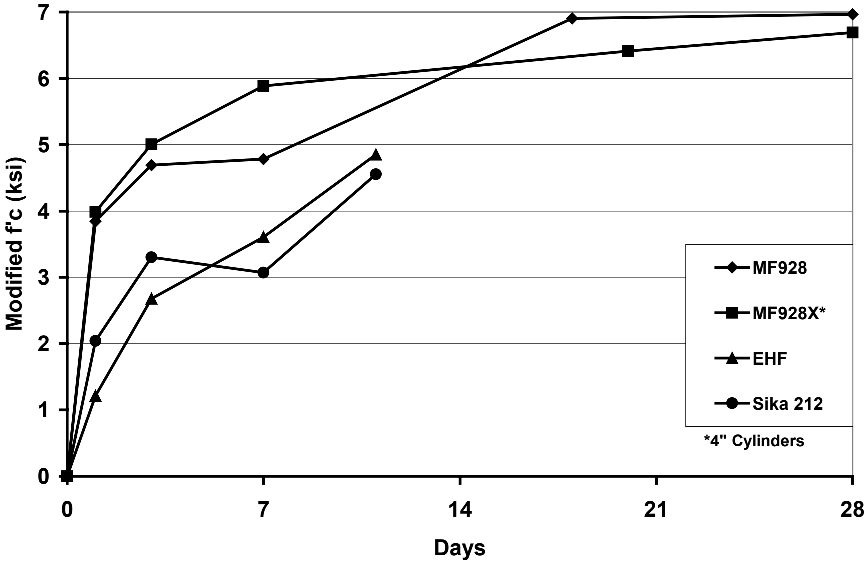


Figure 3.28 Grout Compressive Strengths for Trial Batches

3.2.5 *Instrumentation*

Behavior during pullout tests was determined using both active and passive measurements. Load, strain, and deflection were actively measured using a data acquisition system, which included a Hewlett Packard 3497 scanner and data acquisition software. Load cells were used to measure the load applied to the bar (Figure 3.35). Figure 3.29 shows schematically other active instrumentation used in grout pocket tests.

Connector strain measurements were critical for Phase 1 specimens. Based on the bar strain, the bar force and bond characteristics along the bar length were determined. Strain was typically measured at 6-in. increments along the length of headed bars. As shown in Figure 3.29, strain was measured just beneath the head, at one or more locations along the embedded portion of the bar, and at the lead (i.e., exit) end of the bar. Two strain gages were located at the lead end to monitor bar bending during tests. Strain for straight bars was similarly measured, although strain was usually not measured at the bottom end of the bar, where strain approaches zero.

Strain was measured using 5-mm, quarter-bridge strain gages. Bar preparation included grinding the rebar to a smooth, flat surface, cleaning the surface, epoxying the gage, and waterproofing and protecting the gage with black mastic and foil tape. Care was exercised during grinding to minimize reduction of the cross section of the bar. Figure 3.30 shows a headed bar after strain gage preparation.

For grouted vertical duct specimens, strain gages were additionally placed on vertical ducts to measure the duct strain. As shown in Figure 3.31, strain gages were placed at two orientations, in the circumferential direction and in the direction of the duct seams. Gages were placed circumferentially at 6, 12, and 18 in. from the top of the duct, and one additional gage was oriented along the seam at 12 in.

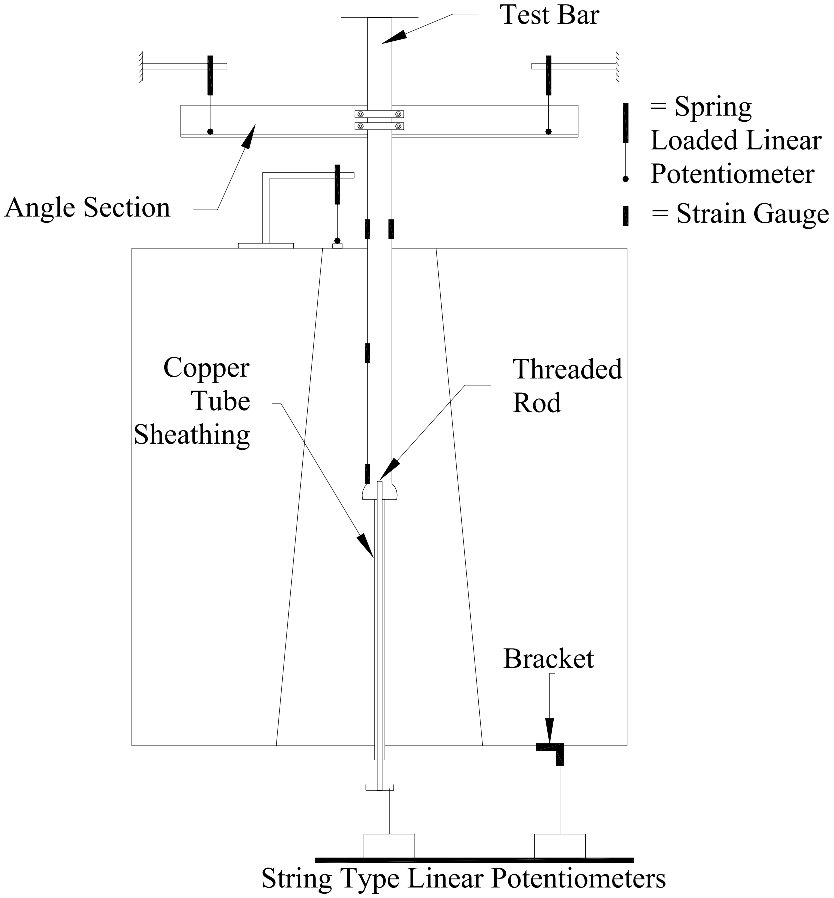


Figure 3.29 Instrumentation Schematic [3.1]

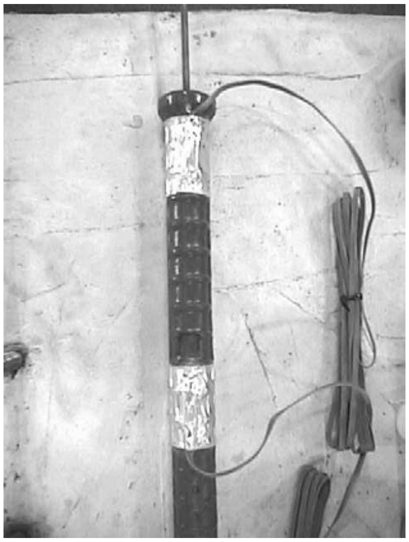


Figure 3.30 Connector after Strain Gage Preparation

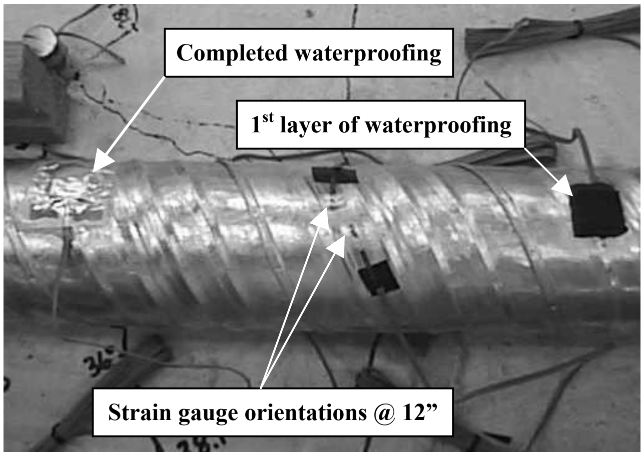


Figure 3.31 Corrugated Duct after Strain Gage Preparation

Deflections were measured for both the bar and concrete beam. Measurements were taken at both ends of a connector to help determine the load-deflection behavior for connectors. Bar deflection, or slip, was measured at the head by drilling and threading a 3/16-in. diameter hole into the head of the bar, screwing a threaded rod into the head, and then extending the rod through the bottom of the beam. A copper tube was used to sheath the rod to prevent bond between the grout and rod. As shown in Figure 3.32, a string-type linear potentiometer was attached to a wing nut threaded onto the bar to measure head deflection. A second linear potentiometer was attached to the underside of the beam to measure the beam deflection during loading. Beam deflection was subtracted from head (and lead) deflections to determine actual deflections.

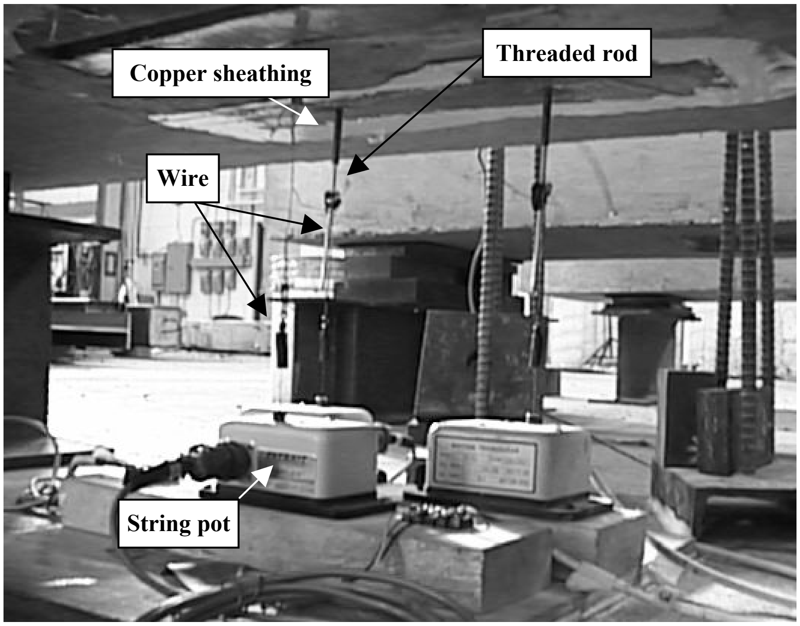


Figure 3.32 String-type Linear Potentiometer for Head and Beam Deflections

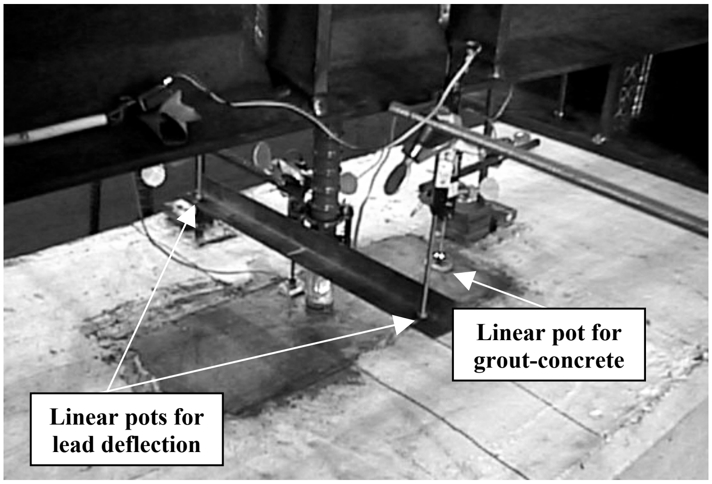


Figure 3.33 Spring-type Linear Potentiometers for Lead Bar and Relative Grout-Concrete Deflections

Lead deflection just above the specimen surface was measured with the aid of a steel angle, as shown in Figure 3.33. Spring-loaded linear potentiometers measured displacement at both ends of the angle. These deflections were averaged to determine the lead deflection in the vertical direction. This allowed any tilt of the connector to be eliminated.

Relative displacement between the grout and concrete was measured to investigate potential slip. This was considered necessary because metal flashing was used in forming grout pocket surfaces. Figure 3.33 shows two linear potentiometer stands located adjacent to a grout pocket. A small Plexiglas square was glued to the grout pocket surface to provide a smooth bearing surface for the linear potentiometers. Similar measurements were taken for both grout pocket and grouted vertical duct tests.

Most active instrumentation provided reliable data. Strain gage records for bars and ducts were found to be especially reliable. Lead displacement records were the most reliable of the linear potentiometers. Grout-concrete measurements were sometimes affected by the formation of grout pocket and beam splitting cracks. String-type linear potentiometers exhibited a particularly large amount of scatter, causing a number of beam and head deflection data records to be erratic. As noted in subsequent sections, in cases where the beam deflection was deemed unreliable, plots of lead and head slip include the beam deflection, and therefore, portray slightly larger displacements. This did not affect any conclusions.

Passive measurements were related to cracks. The specimen was inspected for development and extension of cracks at each load increment. Cracks were marked on the specimen. At several load steps, a crack comparator was used to measure crack widths between 0.002 in. and 0.025 in.

3.2.6 Test Setup

The test setup was designed to efficiently conduct a large number of tests. As shown in the schematic of Figure 3.34, a self-equilibrating test setup was used. This eliminated requirements for tying the specimens to the test floor. Back-to-back channels supported 60-ton center-hole hydraulic rams, 100-kip load cells, a wedge and chuck assembly, hydraulic rams, and load cells (Figure 3.35). Threaded rods connected the channels together and provided a two-inch gap so test bars could extend from the specimen through the center-hole rams. Three short cross beams were used to support the channels on the top of the specimen and were bolted to both the channels and specimen for stability. Crossbeams were positioned approximately 1.5 times the maximum connector embedment depth (i.e., 30 in.) from the bar to prevent confinement effects during testing. Two additional crossbeams were used to support the specimen and provide space for measuring bar head and beam deflections.

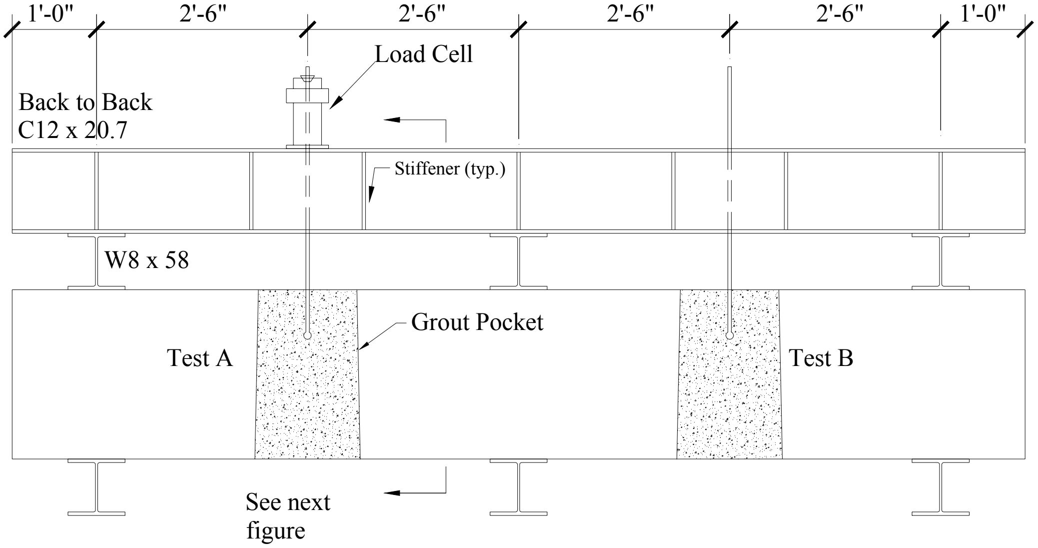


Figure 3.34 Schematic of Typical Test Setup [3.1]

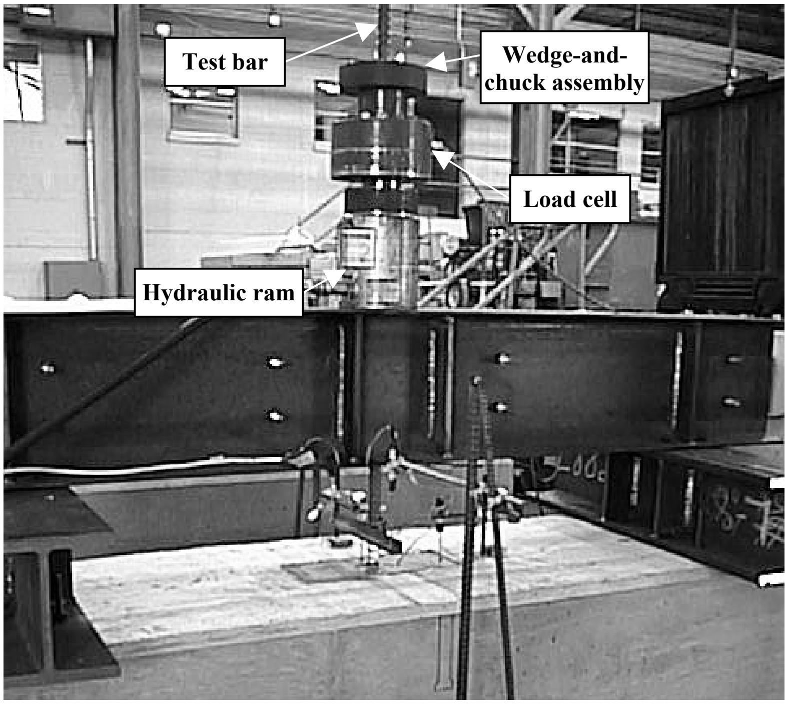
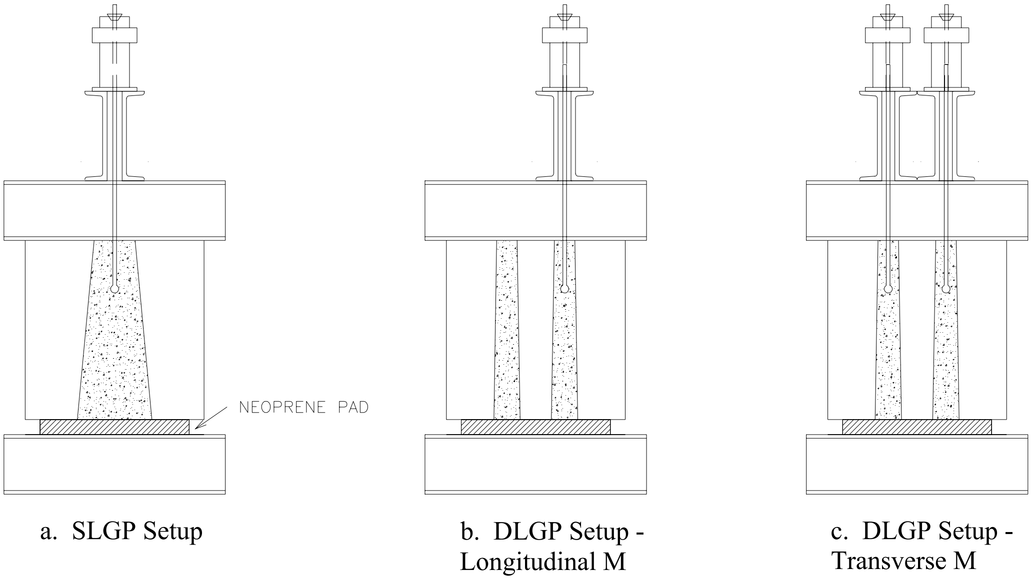


Figure 3.35 Test Setup

Figure 3.36 shows the various ram configurations used for single-line and double-line grout pocket tests. As shown in the figure, tests investigating transverse moment effects required two sets of channels, with one ram on each set. In contrast, tests investigating longitudinal moment or multiple-bar effects required only one set of channels supporting two rams. Figure 3.37 displays the latter case.



a. SLGP Setup

b. DLGP Setup - Longitudinal M

c. DLGP Setup - Transverse M

Figure 3.36 Ram Configuration for Single-Line & Double-Line Grout Pocket Tests [3.1]

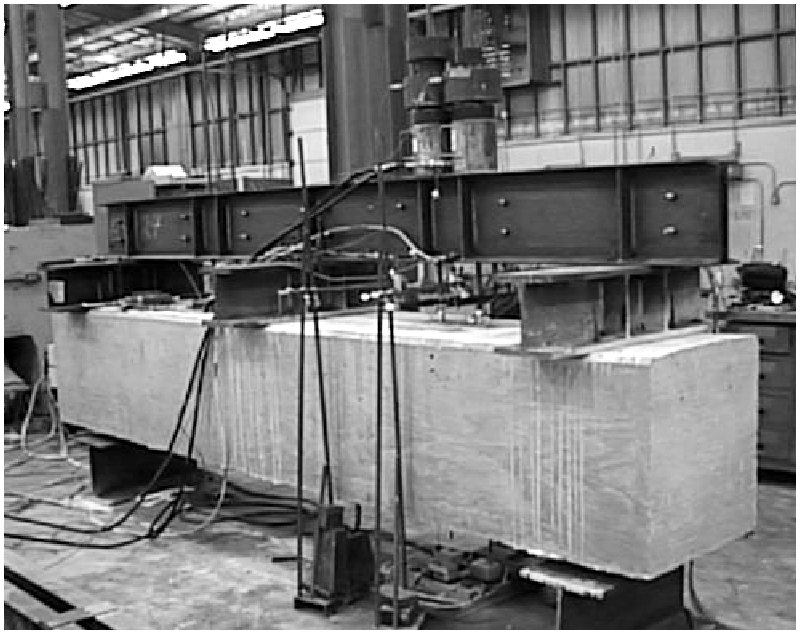


Figure 3.37 Test Setup for Double-Line Grout Pocket Specimen (Longitudinal Moment)

3.2.7 Test Procedure

Testing was conducted by loading the test bar in 2-kip increments with a hand-operated hydraulic pump. As the ram cylinder extended under pressure, the wedge and chuck assembly was pushed upwards, causing the wedges to grip and lift the bar. As shown in Figure 3.35, a load cell was placed between the ram and chuck assembly to measure load applied to the bar.

After each 2-kip increment, the active channels were scanned and the voltage was recorded. Bar load versus lead deflection was plotted during tests to help assess behavior, such as specimen cracking, concrete breakout, and bar yield. Bar strain was also monitored for accidental bending. After scanning the channels, the specimen was inspected for cracks, and cracks were marked and measured. For multiple-bar tests, the bar with the larger load was marked on the specimen. The two loads usually differed by no more than 2%. Specimens were loaded until either a concrete or connector failure was reached. In several cases, loading was discontinued to prevent damage to the load cell or hydraulic hoses, or to avoid fracture of the connector or wedge grips.

A number of anomalies occurred throughout testing, including slip between the wedge grips and the bar, fracture of the grips, and bar bending. In these instances, the specimen was unloaded and corrective measures were taken, such as tightening or replacing wedge grips or re-leveling channels to eliminate bending. After appropriate adjustments were made, loading was resumed.

3.3 SINGLE-LINE GROUT POCKET TESTS

3.3.1 Scope of Testing

Eighteen single-line grout pocket tests were conducted in Phase 1, including fourteen single-bar tests and four double-bar tests. Table 3.6 shows the test matrix for these tests. The first eight single-bar tests were designed to determine major differences in behavior for neat versus extended grouts. These tests were conducted as four pairs, each testing epoxy-coated, #8 upset-headed bars at an embedment depth in the range of 6 in. to 18 in. ($6d_b$ to $18d_b$). The embedment was varied to establish the depth at which the failure mode shifted from concrete breakout to bar yield. The only difference for each pair was the use of neat grout versus grout extended with 3/8-in. pea gravel.

SL09-SL12 tested #6 upset-headed bars to compare anchorage behavior for smaller bars embedded between 4 in. and 8 in. (~5d_b to 11d_b). Based on satisfactory anchorage behavior for neat grouts in SL01-SL08, all subsequent tests used only neat grout. To compare the behavior for different grout brands, SL11 and SL12 used MF928 and EHF, respectively, as the unique variable. SL13 and SL14 used straight bars to compare behavior with tests using headed bars.

After developing an understanding of the failure modes and the effect of the previously mentioned variables on single-bar response, two bars were tested in a single-line pocket in SL15-SL18. SL15 and SL16 provided a direct comparison of the behavior for grouted versus cast-in-place headed bars in an unconfined pocket. SL17 and SL18 investigated the influence of spiral reinforcement and welded wire fabric, respectively, as confining reinforcement.

3.3.2 Summary of Results

Table 3.6 shows the test matrix and select test results for single-line grout pocket tests under three headings: Materials, Load-Deflection, and Cracking. For headed bars, the failure mode was a concrete breakout failure, accompanied by significant cracking in the grout pocket and surrounding concrete and sometimes preceded by bar yield (Figures 3.51a and 3.60c). Straight bars failed by pullout accompanied by splitting cracks and a shallow cone (Figure 3.51b).

Under the Materials heading, the modified grout cube strengths on test day are shown to exceed the concrete cylinder strengths, frequently by 1000 psi or more. Under the Load-Deflection heading are listed the yield load, P_y, and the maximum applied load, P_{max}. Bars that yielded were loaded into the strain-hardening region to try to force a concrete breakout failure. A concrete breakout failure occurred for eight of the fourteen single-bar tests, with three of these being after bar yield. The remaining six single-bar specimens reached yield, but were unloaded before a breakout failure occurred due to test setup limitations, such as grip failure, or to prevent bar fracture.

Yield was achieved in single-bar tests for embedment depths of 12d_b or larger when using #8 bars and for embedment depths of 8d_b or larger for #6 bars. All multiple-bar tests resulted in concrete breakout. The bar slip listed in the table usually corresponds to the head slip at P_{max}, as well as at P_y for bars that yielded.

Load-head slip curves demonstrated stiff, nearly linear response prior to significant cracking, followed by highly non-linear response as the maximum load was approached. Comparison of tabulated values for slip at P_y and P_{max} confirm this. For cases in which head deflection was unreliable, lead deflection at P_{max} or P_y is listed to provide some measure of stiffness up to yield. Lead slip typically demonstrated a softer response, as the effects of bond degradation along the bar were included.

Under the Cracking heading, the load at which splitting cracks at the connector first appeared on the grout surface, P_{split}, is listed, together with the corresponding crack width. It is important to note that splitting cracks passed through the connector at early stages of loading, which is expected to reduce connector capacity (Reference 3.4). The maximum surface crack width at a range of service level loads (e.g., 60%-80% of P_{max} or P_y) is also listed. Surface crack widths usually decreased for greater embedment depths. Confined specimens exhibited a better distribution of cracking and smaller crack widths overall.

Table 3.6 Single-Line Grout Pocket Test Matrix and Select Results

						Materials			Load-Deflection			Cracking			
Test		**Bars**			**hef**	**Grout**	**f'c (ksi)**		**Pyield^{F}**	**Pmax^{F}**	**Slip^{C}**	**Psplit^{F}**	**width**	**Pserv^{F}**	**width**
ID	**type**	**no.-size**	**anchor**	**confining**	**in.**	**brand**	**grout^{E}**	**concr**	**kips**	**kips**	**10^{-3} in.**	**kips**	**10^{-3} in.**	**kips**	**10^{-3} in.**
SL01	SLGP	1-#8	upset head	n/a	6	MF928	6.9	5.4	*	36	38	12	13	21	16
SL02	SLGP	1-#8	upset head	n/a	6	MF928X	6.4	5.4	*	37	34	15	2	21	3
SL03	SLGP	1-#8	upset head	n/a	12	MF928	6.5	5.2	53	60^{A}	17, 68	31	9	37	9
SL04	SLGP	1-#8	upset head	n/a	12	MF928X	6.3	5.4	55	63^{A}	11, 48	44	3	44	3
SL05	SLGP	1-#8	upset head	n/a	9	MF928	7.0	6.3	*	46	46	21	5	24	5
SL06	SLGP	1-#8	upset head	n/a	9	MF928X	7.1	6.5	*	45	47	30	5	30	5
SL07	SLGP	1-#8	upset head	n/a	18	MF928	7.0	5.4	53	70^{B}	10, 25	17	3	29	3
SL08	SLGP	1-#8	upset head	n/a	18	MF928X	8.5	5.5	53	64^{B}	5, 15	30	2	30	2
SL09	SLGP	1-#6	upset head	n/a	4	MF928	7.4	5.5	*	21	176	9	10	9	10
SL10	SLGP	1-#6	upset head	n/a	8	MF928	7.5	5.3	28	40^{B}	$<39^{D}$. -	12	13	18	13
SL11	SLGP	1-#6	upset head	n/a	6	MF928	6.9	5.2	26	34^{A}	$<60^{D}$. -	8	13	19	16
SL12	SLGP	1-#6	upset head	n/a	6	EHF	6.3	5.2	29	35^{B}	$<53^{D}$. -	12	13	23	13
SL13	SLGP	1-#8	straight	n/a	12	MF928	6.9	5.2	52	56^{B}	$<37^{D}$. -	8	2	44	13
SL14	SLGP	1-#8	straight	n/a	18	MF928	6.9	5.1	52	73^{B}	$<37^{D}$. -	6	2	40	13
SL15	SLGP	2-#8	upset head	n/a	12	MF928	5.3	5.0	*	32	41	12	2	26	5
SL16	CIP	2-#8	upset head	n/a	12	n/a	n/a	5.0	*	39	$<66^{D}$	18	2	32	5
SL17	SLGP	2-#8	upset head	spiral	12	MF928	6.2	4.9	*	48	91	16	2	32	5
SL18	SLGP	2-#8	upset head	wwf	12	MF928	5.6	5.4	*	49	115	18	2	24	7

Footnotes

A. Concrete breakout failure after bar yield

B. Test stopped due to limitations of test setup or to prevent bar fracture

C. Slip corresponds to head slip at yield and/or maximum load, except as noted

D. Based on lead deflection; head deflection unreliable

E. Grout cube strength modified by 0.8 factor

F. Load shown is the applied load per bar

Abbreviations

*=Concrete breakout before bar yield

MF928=Masterflow 928, neat

MF928X=Masterflow 928 extended with 3/8" pea gravel

EHF=Euclid Hi-Flow, neat

3.3.3 General Behavior

3.3.3.1 Overview

Pullout behavior for an upset-headed bar grouted in a single-line grout pocket exhibited four main stages of response: 1) anchorage by adhesion, friction, and bearing along bar deformations, 2) formation of splitting cracks in the grout pocket, initiating a load-sharing between the upset head and the bar shaft, 3) formation and extension of splitting cracks in the surrounding beam and development of cracks at the grout pocket corners, causing a loss of pocket confinement, and 4) bar yield and/or concrete breakout failure.

Similar to a regular deformed bar, upset-headed bars initially achieve anchorage by adhesion and friction, although this is quickly lost due to the Poisson effect and the use of epoxy coating. Bond is then transferred primarily by bearing on bar deformations, with only a small initial load transfer at the upset head. As the applied load is increased, circumferential tensile stresses due to bearing on the lugs and upset head produce splitting cracks in the grout along a partial length of the bar. Tensile stresses required for equilibrium of the test specimen also contribute to the formation of these cracks. For shallow embedment depths (e.g., $9d_b$ or less), splitting may initiate at the head and extend along the length of the bar (Section 3.5.4.3), although in most cases cracks do not appear at the grout surface until larger loads are applied. These splitting cracks are arrested by the confinement effects of the concrete and longitudinal bars around the grout pocket. However, the formation of splitting cracks causes deterioration of bond along the length of the connector, transferring more load to the head and reducing anchorage stiffness.

As the applied load is increased, splitting cracks extend into the surrounding concrete and new cracks emanate from the grout pocket corners into the concrete. Depending on embedment depth, connectors may yield before or after corner cracks develop. Corner cracks effectively divide the specimen into four distinct tensile stress fields. Stress concentrations at these reentrant corners and wedging action due to the shape of the grout pocket contribute to the formation and growth of these cracks. This reduces confinement of the grout pocket, causing a further degradation of bond along the bar, which sheds more load to the head. Corner cracks then combine with failure planes that extend primarily from the bar head to produce a concrete breakout failure on the two sides of the beam. This is accompanied by shallow surface cones that spall above the longitudinal reinforcement at each end of the grout pocket.

3.3.3.2 Illustration

As an illustration of this general behavior, which was observed for most single-line grout pocket tests, the response of specimen SL03 is described. SL03 used a single #8 upset-headed bar grouted 12 in. in a single-line pocket with MF928 neat grout. Although an actual precast bent cap system allows for a much deeper embedment and involves a more complex force distribution, the use of neat grout and achievement of bar yield in SL03 provides a meaningful illustration.

Load-slip response and the distribution of forces between the shaft and the head are portrayed by Figures 3.38 and 3.39, respectively. Figure 3.38 plots applied load, P, versus slip at the bar head and lead end. Figure 3.39 plots bar force at the head versus applied load. Bar force at the head of the bar is based on the measured strain adjacent to the upset head (Figure 3.30). As shown in Figure 3.39, the vertical distance between the data plot and the diagonal line represents the portion of applied load transferred by bond, whereas the vertical distance between the horizontal axis and the data represents the load carried by the upset head.

Initially, the applied load was transferred to the surrounding grout and concrete primarily by adhesion and bearing along the bar deformations. This is shown by the fact that the head force remains at zero up to a load of 17 kips. Figure 3.39 shows a sudden 5-kip increase in the head force during the next load step, corresponding to the formation of splitting cracks. The development of splitting cracks at this load step caused a transfer of 30% of applied load from the shaft to the head. Depending on the embedment depth, splitting initiates at the head or occurs there shortly after initial splitting, due to the loss of bond and greater stiffness and subsequent force transfer at the head.

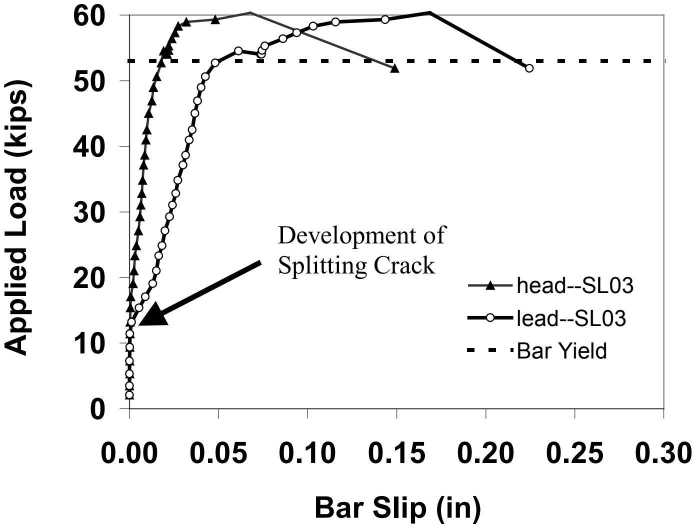


Figure 3.38 Load-Bar Slip Response (SL03)

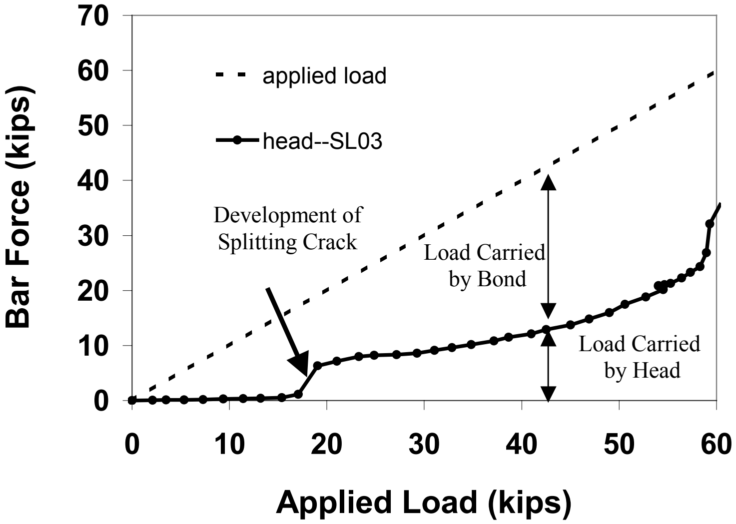


Figure 3.39 Distribution of Bar Force (SL03)

Although splitting cracks did not immediately appear at the grout surface, both displacement and strain records indicate that splitting occurred. At 17 kips, the load-slip plot shows a sudden reduction in stiffness for both the head and lead locations. The larger reduction in stiffness for the lead end reflects the loss of bond along the shaft that accompanied cracking of the grout. Loss of bond for a #8 bar embedded 12 in. could increase the measured lead slip by as much as 0.03 in. At 31 kips, the splitting crack first appeared at the grout surface, extending across the narrow width of the pocket through the bar (Figure 3.40) and opening to a width of 0.009 in.

Figure 3.40 Splitting Crack in Grout Pocket (SL03)

The confining effect of the surrounding concrete limited formation of splitting planes in the concrete until a load of 45 kips. From first splitting to 45 kips, load-slip response exhibited a nearly constant stiffness, and the gradual increase in head force reflects the moderate deterioration of bond due to the effectiveness of the concrete in confining the grout pocket. At 45 kips, splitting cracks appeared on the top and side surfaces of the concrete beam, as shown in Figure 3.41. Splitting cracks on the surfaces of the side face extended to about half the depth of the connector. Although steel confinement around the pocket was not employed, the longitudinal top bars provided some measure of confinement and limited the opening of splitting cracks.

At 51 kips, grout pocket corner cracks appeared at the surface. Upon further loading, these cracks gradually extended toward the sides of the beam, as shown in Figure 3.42. In addition, side face splitting cracks extended beyond the embedment depth. These two types of cracks substantially reduced the effectiveness of the concrete in confining the grout pocket, resulting in the softening shown in Figure 3.38. At 53 kips, the bar yielded. This is portrayed clearly in Figure 3.38, where the lead deflection increased suddenly. Head slip at this load was only 0.017 in., indicating a stiff overall response to yield.

The load increased further as the ram was pumped repeatedly to stretch the bar into the strain-hardening region. A concrete breakout failure occurred at a load of 60 kips, as shown in Figures 3.43 and 3.44. This shape shows semblance of a concrete breakout surface with edge effects on both sides of the beam, although corner cracks divided the tensile stress field into four regions, one on each side of the pocket. Figure 3.44 shows spalling of shallow cones at the ends of the grout pocket. These sides of the pocket were less restrained, as longitudinal bars passed adjacent to the exterior sides of the pocket. Removal of the top layers of grout and the shallow cones revealed that cracks initiated at the bar lugs and extended through the grout into the surrounding concrete. These cracks intersected with grout pocket corner cracks to produce the spalled concrete shapes shown.

Between loads of 53 kips and 60 kips, the load-slip response of the head became highly nonlinear, indicating mobilization of the failure surfaces. Figure 3.39 shows a rapid increase in load at the head as a further loss of bond and surface spalling occurred near failure. At failure, the head carried approximately 60% of the applied load.

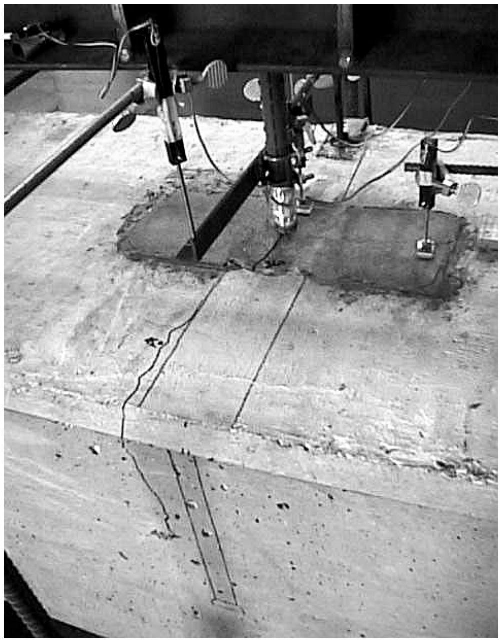


Figure 3.41 Extension of Splitting Crack into Surrounding Concrete (SL03)

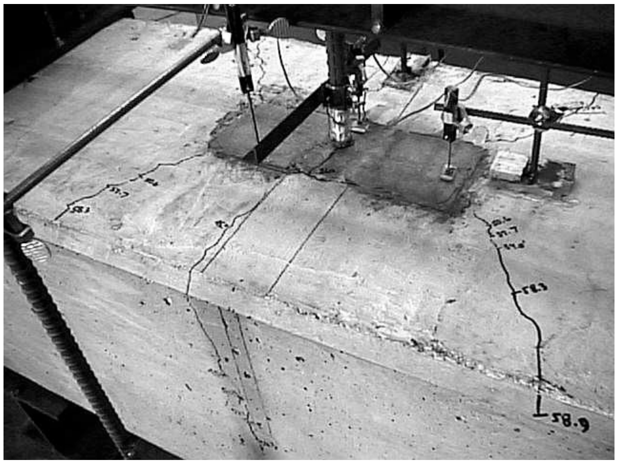


Figure 3.42 Grout Pocket Corner Cracks (SL03)

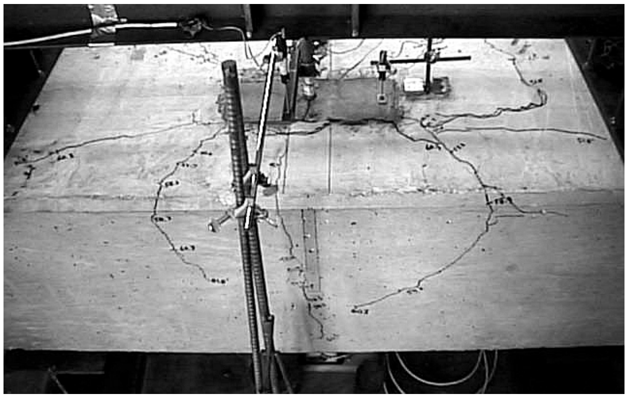


Figure 3.43 Cone Breakout Failure (SL03)

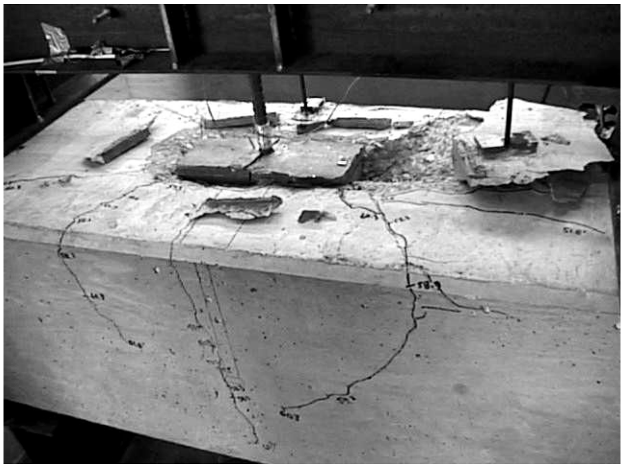


Figure 3.44 Spalling of Shallow Cones Adjacent to Pockets (SL03)

Relative movement at the grout-concrete interface was negligible, despite the use of smooth forms during casting. Figure 3.45 shows movement between the concrete and pocket at the splitting load. The larger displacement at the west side of the pocket reflects the formation of a splitting crack directly under the gage. Although relative movement increased at both locations with loading, this response reflected an overall movement of the grout surface related to splitting and spalling, not relative slip at the interface. Excellent interlock developed between the pocket and concrete.

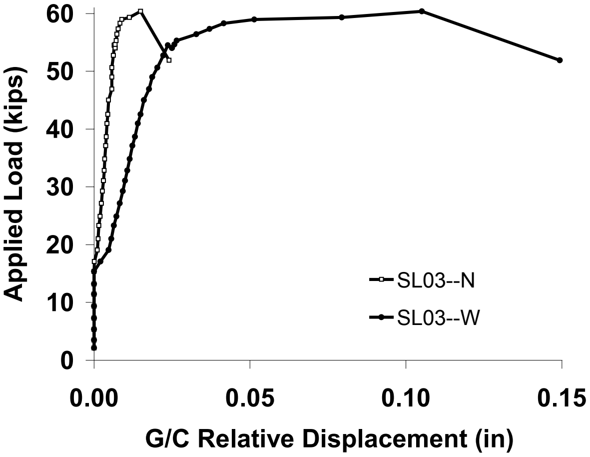


Figure 3.45 Relative Grout-Concrete Displacement Response (SL03)

3.3.4 Effects of Variables

3.3.4.1 Embedment Depth

Single-bar tests, SL01-SL14, were designed to investigate changes in behavior as the embedment depth increased. Of particular interest was the depth at which a transition between concrete breakout and bar yield occurred. Figure 3.46 shows the increase in maximum applied load with increasing embedment depth for #8 headed and straight bars and #6 headed bars. Bar yield was achieved for #8 bars at an embedment of 12 in. ($12d_b$) or larger. Number 6 bars yielded at an embedment depth of 6 in. ($8d_b$). Bars reached yield at embedment depths substantially less than predicted by Chapter 12 of ACI 318-99 for straight bar anchorage (Reference 3.5). Concrete breakout, however, may still occur at loads only slightly larger than P_y. For example, SL03 and SL04 exhibited concrete breakout failure at a load of $1.15P_y$.

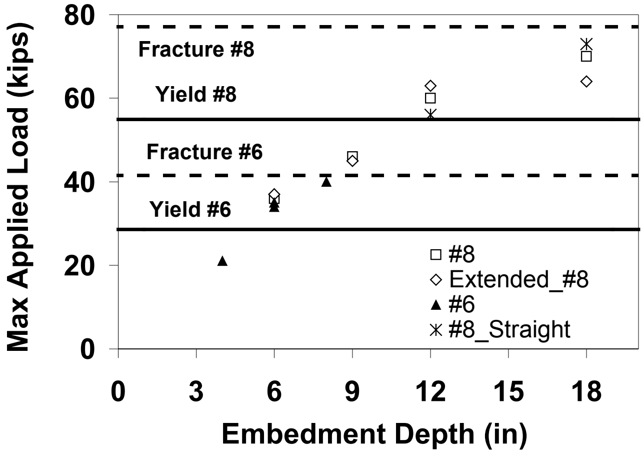


Figure 3.46 Relationship between Applied Load and Embedment Depth for #8 and #6 Bars

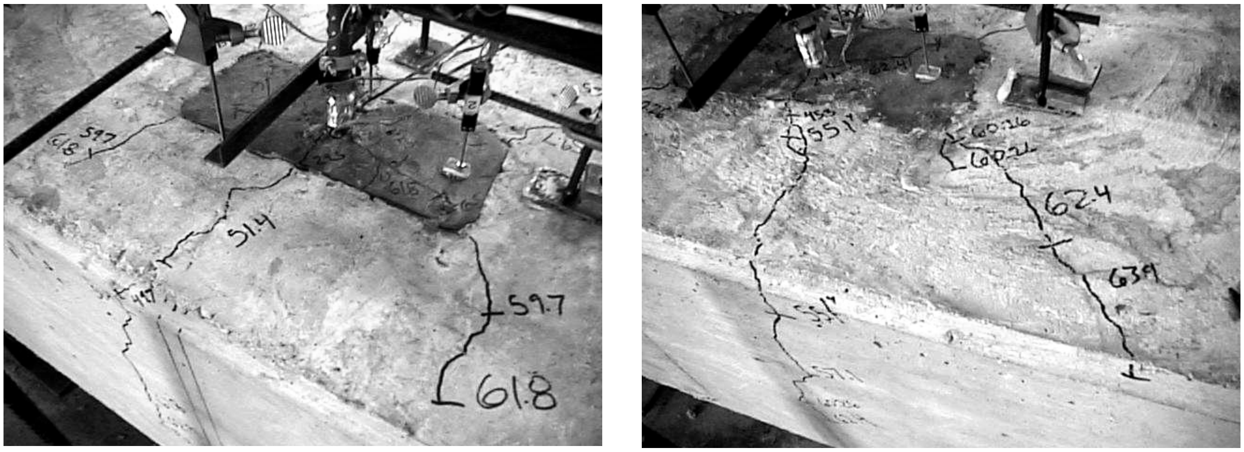
In addition to providing sufficient anchorage for bar yield, greater embedment depths enabled load transfer to occur over a greater depth of the cross section. This resulted in a better distribution of bond, a smaller influence of splitting cracks on bond deterioration, and reduced the force carried by the upset head. For example, the head carried 60% of the load for a 12-in. embedment, but less than 25% for an 18-in. embedment.

Table 3.6 also shows measured crack widths at the grout pocket surface for loads in the service-level range. Surface crack widths generally decreased with increased embedment depth. This was particularly noticeable for the 18-in. embedment. However, surface crack widths do not have a direct relationship to actual bent cap service load conditions, since the grout surface for pullout tests is not an exposed surface in an actual connection and the actual transfer of forces in a connection is more complex. Nevertheless, crack measurements still provide some indication of the potential level of cracking within a connection related to bar anchorage.

3.3.4.2 Grout Extension

The first eight tests were designed to compare the performance of grout pockets in four pairs using neat MF928 grout and MF928 grout extended with 3/8-in. pea gravel. Table 3.6 shows that nearly identical maximum loads were achieved for all pairs. In addition, relative slip between the grout and concrete, which was a concern prior to testing, was negligible for both cases. Figures 3.47a and 3.47b show the similarity in specimen cracking for the SL07-SL08 pair. SL07 used neat MF928 grout, whereas SL08 used extended MF928 grout, both with a #8 upset-headed bar embedded 18 in.

However, there were two differences in response: 1) sudden bar slip when splitting cracks formed in neat grouts, and 2) softer head response for neat grouts. Figure 3.48 compares the load-slip behavior for SL07-SL08. At a load of 15 kips, the lead deflection for SL07 showed a sudden increase, due to the formation of splitting cracks and subsequent loss of bond. This type of behavior was evident in all connections that used neat grouts. In contrast, SL08 and other tests that used extended grouts showed little to no increase in slip and only a minor increase in bar force when splitting cracks formed. The presence of coarse aggregate tends to reduce, but not eliminate, the effects of cracking in the grout matrix. In addition, a stiffer response for extended grouts is evident, due to the contribution of coarse aggregate to the grout matrix stiffness. Figure 3.48 shows that the extended grout exhibited a response ~70% stiffer than the neat grout at yield (a small fraction of this is attributed to the larger test-day grout cube strength). Nevertheless, specimens with neat grout still demonstrated adequate overall stiffness and strength. Because of significant constructability advantages, neat grout was used in all tests after SL08.



a. Neat MF928 *b. Extended MF928*

Figure 3.47 Crack Pattern for Neat and Extended Grouts (SL07 and SL08)

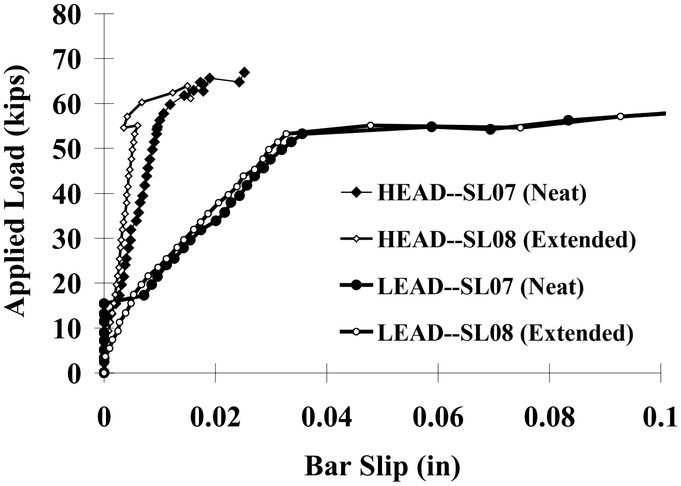


Figure 3.48 Load -Slip Behavior for Neat and Extended Grouts (SL07 and SL08)

3.3.4.3 Grout Brand

SL11 and SL12 compared response for single-line pockets grouted with MF928 and EHF, respectively, using upset-headed bars embedded 6 in. Although considerable differences were observed during and after grouting operations, pullout strength and stiffness compared closely. This is shown in Figure 3.49, which plots load versus lead displacement (measurements at the head were unreliable). Other than the sudden increase in lead displacement for MF928 upon splitting at 8 kips, behavior was very similar up to failure. Similar crack patterns and crack widths developed as well, although EHF exhibited more cracking in the grout pocket around the bar at failure (Figure 3.50). Overall, the load transfer performance for both grouts was acceptable.

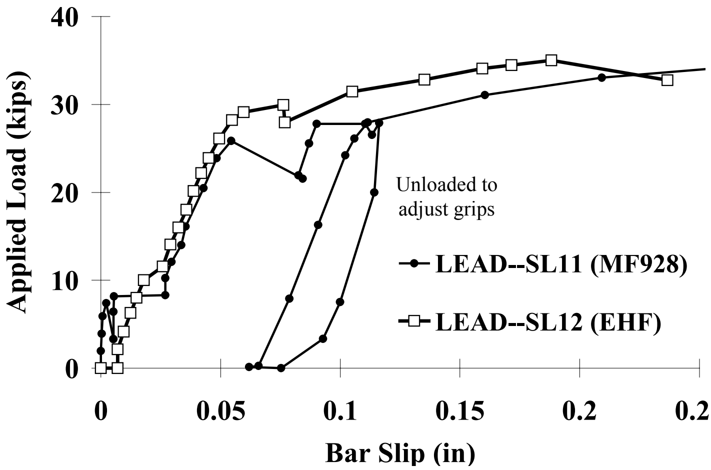
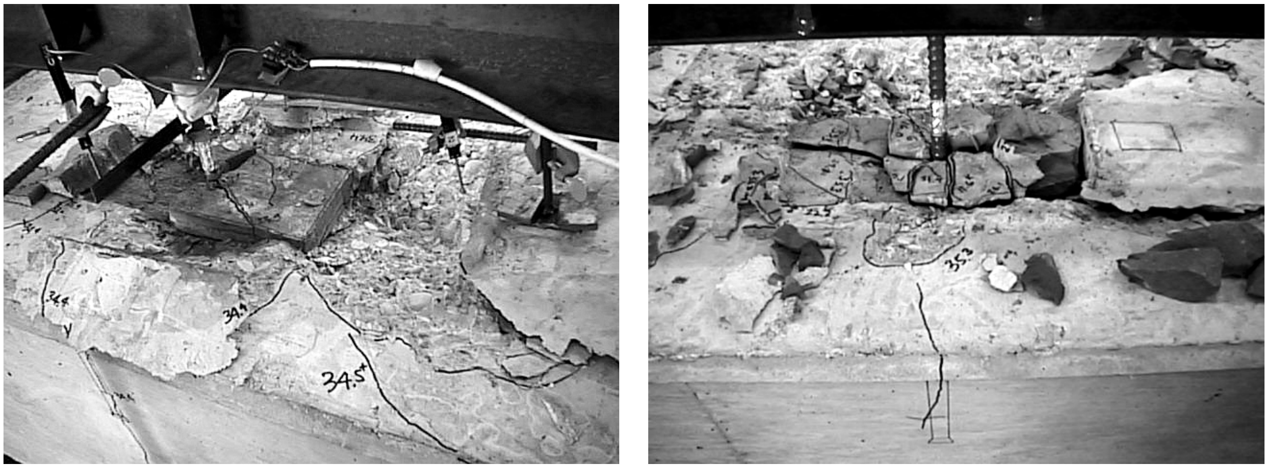


Figure 3.49 Load-Slip Behavior for MF928 and EHF (SL11 and SL12)

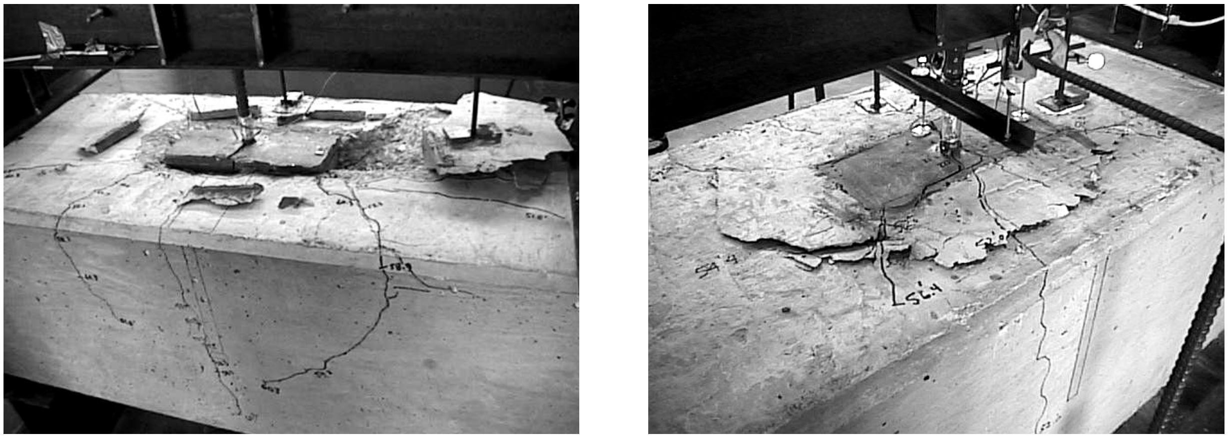


a. Masterflow 928 *b. Euclid High Flow*

Figure 3.50 Grout Pocket at Failure for MF928 and EHF (SL11 and SL12)

3.3.4.4 Bar Anchorage

Two tests, SL13 and SL14, were conducted to determine pullout behavior of straight bars grouted in single-line pockets. These tests used embedment depths of 12 in. and 18 in., respectively, and thus directly compare with SL03 and SL07, which used upset-headed bars at the same respective depths. Straight bar response appeared similar to headed bar response prior to failure, with splitting cracks extending across the grout pocket and into the concrete, followed by the appearance of grout pocket corner cracks at the time of bar yield. However, as shown in Figure 3.51b, a shallow cone and additional radial cracks accompanied pullout of the bar at failure, rather than the concrete breakout surface characteristic of headed bars such as those shown in Figures 3.51a and 3.60c.



a. Headed Bar *b. Straight Bar*

Figure 3.51 Specimen Failure for Headed and Straight Bars (SL03 and SL13)

Figure 3.52 compares load-lead slip behavior for SL03 and SL13. Surprisingly, the straight bar shows a stiffer response to yield after both bars slipped as a results of the formation of splitting cracks. A similar behavior was observed for straight and headed bars grouted in vertical ducts (Figure 3.88). Bar force distribution for the straight bar (Figure 3.53) shows how splitting cracks at 11 kips caused an almost complete loss of bond along the top half of the straight bar. However, at 20 kips, the straight bar re-engaged the concrete. This indicates that, following bar slip, the interlock of the bar within the pocket became effective again. This may be attributed to the wedge-shaped grout pocket that mobilizes confinement of the surrounding concrete after a slight vertical displacement. Due to confining effects, the #8 straight bar was developed in just $12d_b$. Similar response resulted for the straight bar embedded 18 in. It should be noted, however, that alignment difficulties in the test setup caused bars in SL13 and SL14 to bend during tests, with strain differentials as large as 1500 microstrain between the two lead strain gages. The degree to which this may have influenced results is unclear. Straight bars tested in Phase 2 and 3 connections achieved yield using an embedment of $13d_b$.

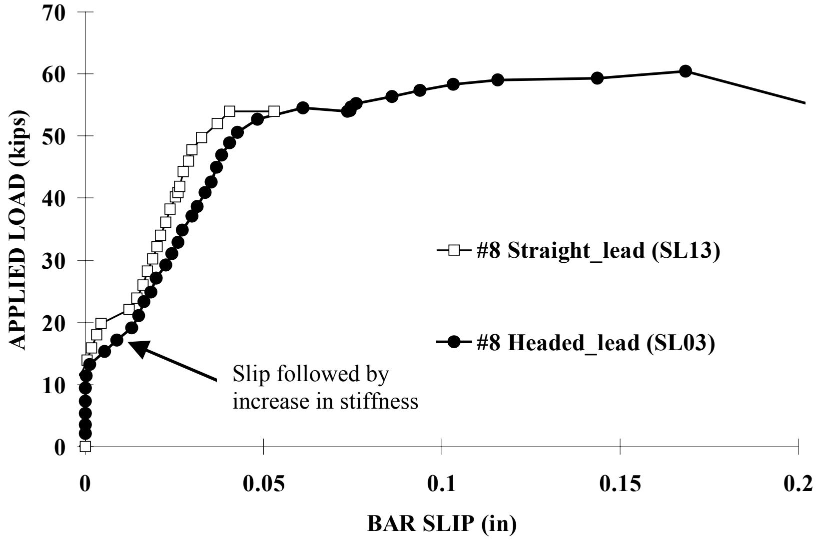


Figure 3.52 Load-Slip Behavior for Headed and Straight Bars (SL03 and SL13)

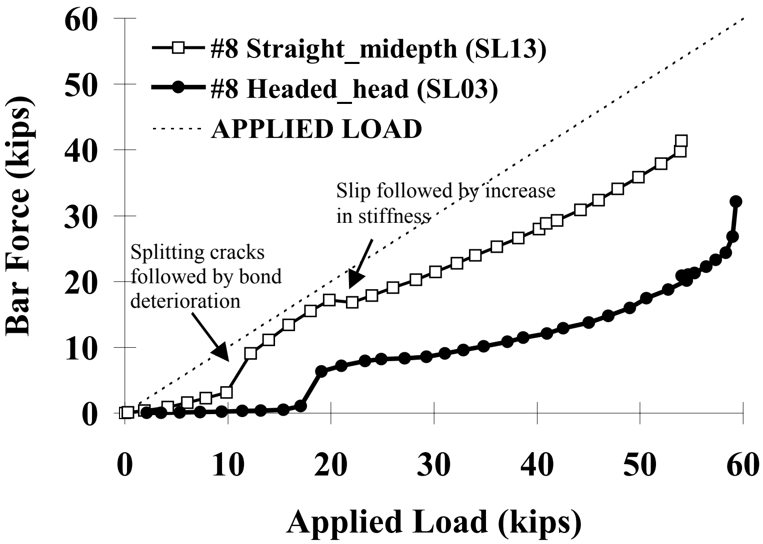
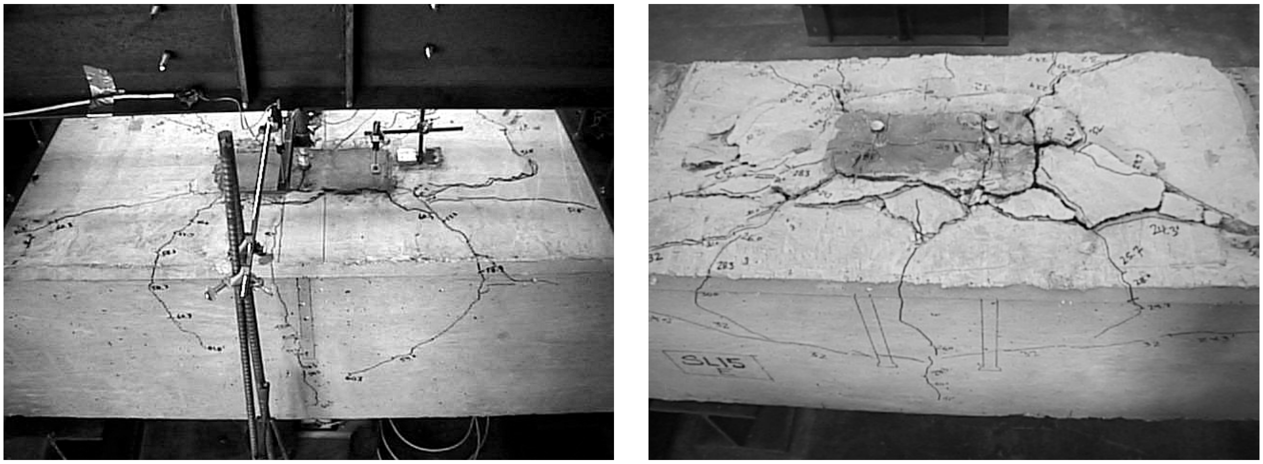


Figure 3.53 Distribution of Bar Forces for Headed and Straight Bars (SL03 and SL13)

3.3.4.5 Multiple Bars

Beginning with SL15, two epoxy-coated upset-headed #8 bars were tested, each at an embedment of 12 in. SL15 may be compared to SL03, which used a single upset-headed #8 bar at the same embedment depth. The general overall behavior was similar for the two cases. Both specimens eventually failed by concrete breakout, and the capacity of SL15 was only 6% larger than SL03. The single bar loaded in SL03 achieved yield, while the SL15 bars did not, as the load per bar was half that in the bar for SL03. The failure surface that appeared on the east side face of SL03 resembled that of SL15, as shown in Figure 3.54. Figure 3.55c shows that the failure surface for SL15 spreads out further along the beam than SL03. In Chapter 6, actual capacities are compared to predicted capacities based on the Concrete Capacity Design Method.



a. Single Bar Loading *b. Multiple Bar Loading*

Figure 3.54 Failure Surfaces for Single Bar and Multiple Bar Loading (SL03 and SL15)

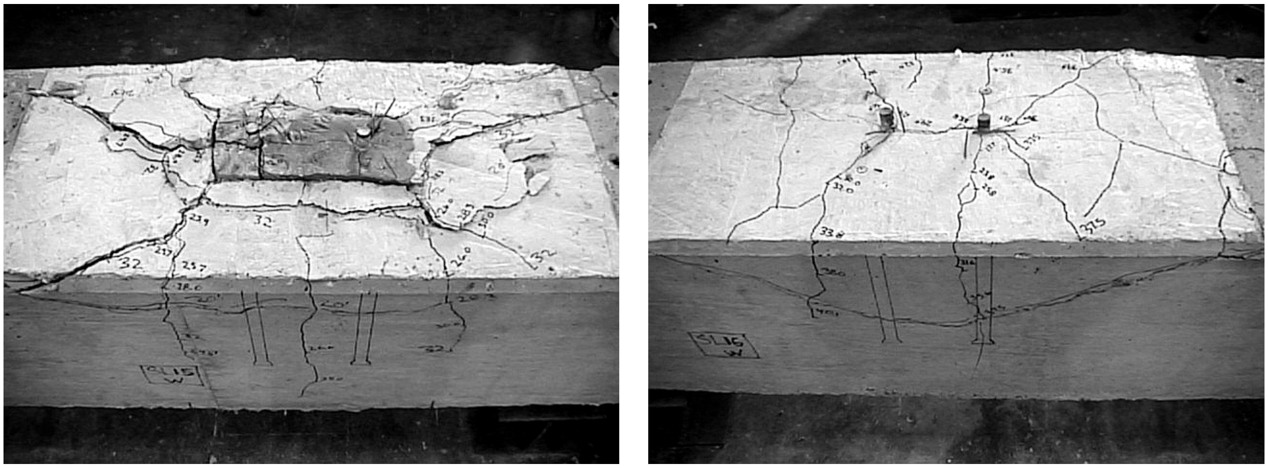
3.3.4.6 Grouted Versus Cast-in-Place Bars

SL15 and SL16 compared behavior for specimens using two upset-headed #8 bars that were grouted versus cast-in-place. The response of cast-in-place multiple bars in SL16 differed significantly from the grouted bar response. The primary failure surface, which appeared suddenly, was a breakout cone, defined by a wide continuous cone-shaped crack that extended from the bar heads on both sides of the beam (Figures 3.55b-3.55d). The location of the crack reveals that the failure surface developed at an angle of approximately 20 degrees along the beam. The various stages of response typical of grouted pockets were not evident, and splitting cracks caused only minor effects.

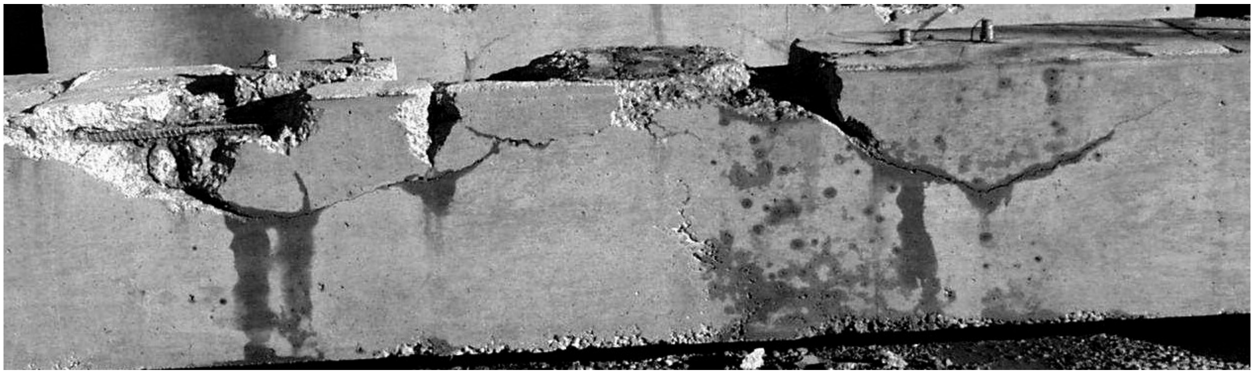
Despite some similarities in crack patterns shown in Figures 3.54b, 3.55b, and 3.55c, behavior of SL15 and SL16 was dissimilar (Figures 3.55a and 3.55d). Figure 3.55a shows deep and wide grout pocket corner cracks, which led to a very disturbed region around the pocket and eventual spalling of much of the top surface (Figure 3.55d). Figures 3.55c and 3.55d show the specimens after a small amount of jackhammering to remove loose concrete and expose failure surfaces.

Data records also clearly indicate dissimilar behavior. Figure 3.56 shows the stiffer response of the cast-in-place specimen up to the failure load, followed by a sudden loss of load. SL15 shows a markedly softer load-slip behavior, reflecting the gradual development of cracks and loss of confinement from the surrounding concrete. The capacity was about 20% less than for the cast-in-place connectors. This may be explained by the greater disturbance of the tensile stress fields for grouted anchors compared to cast-in-place anchors. Figure 3.57 shows an idealization of tensile stress fields for the two cases, ignoring splitting cracks. Reference 3.4 reported that, depending on crack width, 25% to 45% lower failure loads result for single anchors in concrete when the tensile stress state is disturbed by cracks in the load-transfer area. This is due to the decreased surface area available for transfer of tensile forces. Based on tests of multiple anchors in tension, Reference 3.4 concluded that the maximum group strength reduction is approximately equal to the strength reduction for a single anchor in a crack, and that group performance is affected only slightly by the position of anchors relative to multiple cracks. Results for SL15 and SL16 are consistent with these findings, based on the fact that splitting cracks in SL16 did not appreciably disturb the load transfer area.

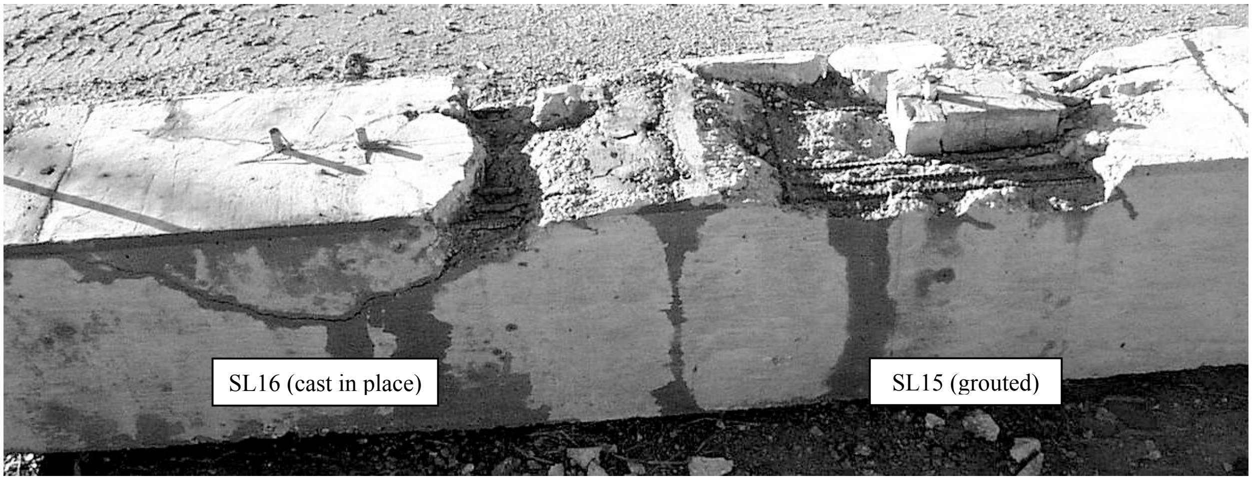
A comparison of the distribution of forces is shown in Figure 3.58. A significantly larger head force for the grouted case was observed, especially as failure was approached. In addition, the effects of splitting are more pronounced for the grouted case, especially at shallower depths. The more moderate bond deterioration for cast-in-place bars may be attributed to the presence of coarse aggregate. A similar effect was noted earlier when comparing neat and extended grouts.



a. SL15 (Grouted)　　*b. SL16 (Cast in Place)*



c. Failure Surfaces for Grouted and Cast-in-place Specimens



d. Failure Surfaces for Grouted and Cast-in-place Specimens

Figure 3.55 Comparison of Failure Surfaces for Grouted and Cast-in-place Specimens

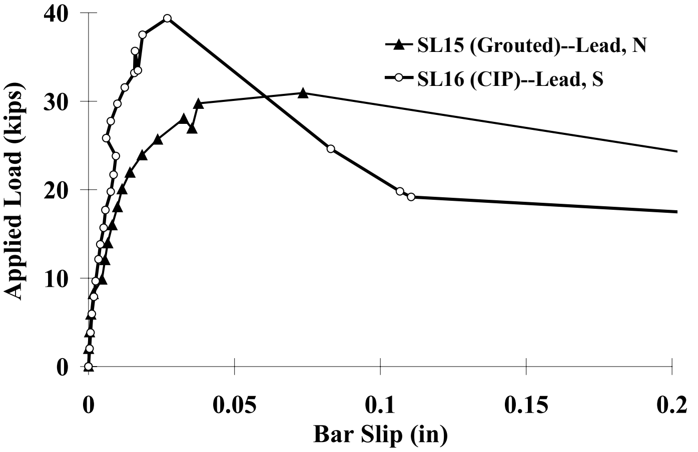
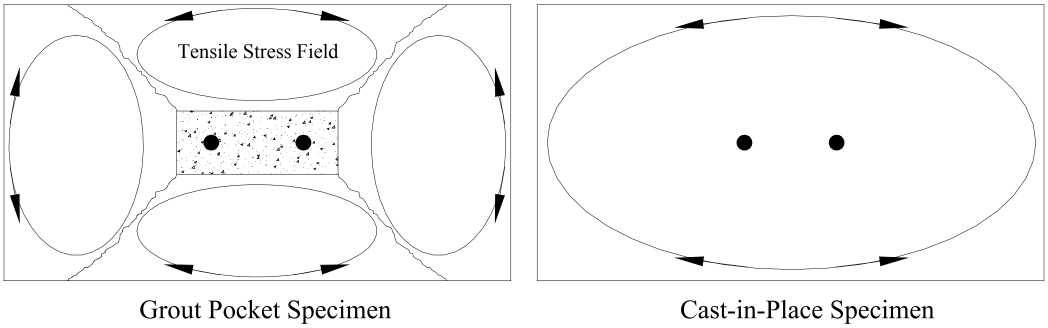


Figure 3.56 Load-Slip Behavior for Grouted and Cast-in-place Bars (SL15 and SL16)



Grout Pocket Specimen Cast-in-Place Specimen

Figure 3.57 Idealized Tensile Stress Fields for Grout Pocket and Cast-in-place Specimen (compression struts not shown) [3.1]

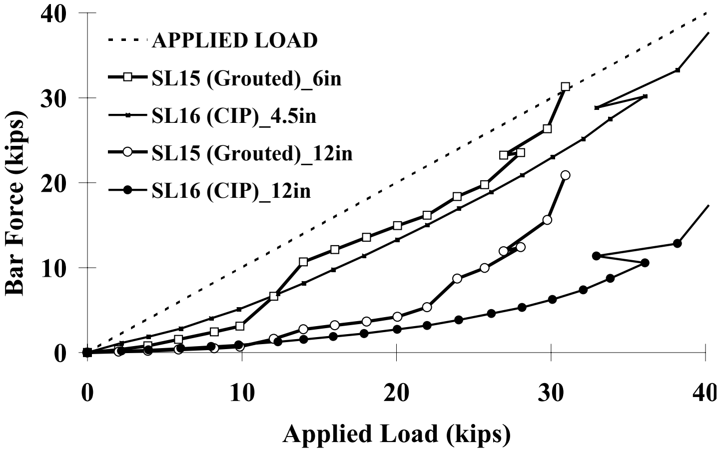


Figure 3.58 Distribution of Bar Forces for Headed and Straight Bars (SL03 and SL13)

3.3.4.7 Confinement

Confinement effects for multiple-bar tests were investigated in SL17 and SL18. Test parameters were the same as SL15, except that spiral reinforcement and welded wire fabric were used to confine the SL17 and SL18 grout pocket regions, respectively. Similar behavior for all three tests was observed prior to the formation of splitting cracks in the concrete. Upon splitting, confining reinforcement arrested crack growth and crack widths, especially for SL17, and caused a better distribution of cracks. As shown in Figure 3.59, confined specimens achieved a much larger capacity and ductility compared to the unconfined case. SL15 did sustain load for an increment beyond peak load, although post-peak behavior was essentially brittle. SL17 and SL18 achieved maximum loads 50% larger and maximum lead slips at least 2.5 times larger than the unconfined case. The use of confining reinforcement did not enable the peak load to be sustained, although load dropped off much more gradually after the maximum load was reached. Because of this drop off, maximum lead displacements are compared at the load corresponding with the maximum load reached by SLl5.

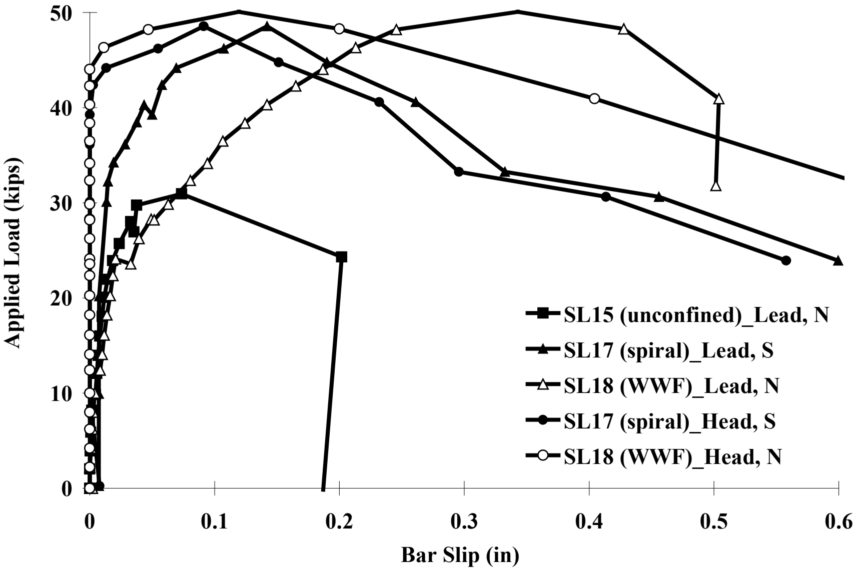
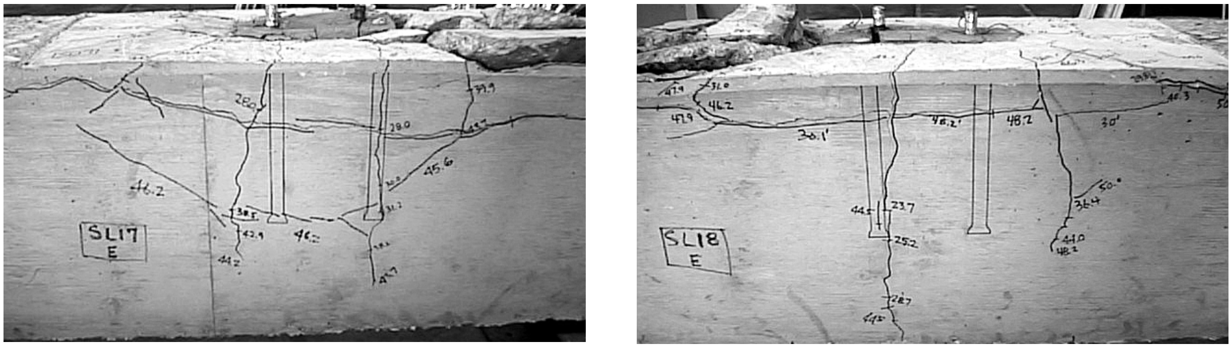


Figure 3.59 Load-Slip Behavior for Unconfined and Confined specimens (SL15 vs. SL17 and SL18)

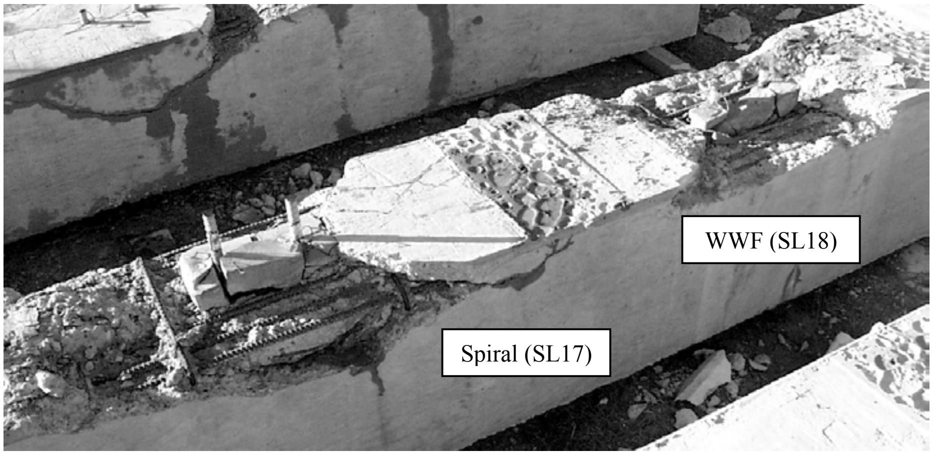
A cone breakout surface appeared on the west side of SL17 at a load of 44 to 46 kips (Figure 3.60), but required additional deformation to achieve the remaining 10% of the capacity. As shown in Figure 3.61, the load distribution diagram showed a sudden decrease in head load and corresponding increase in bond between 6 in. and 12 in. at loads larger than 44 kips for both SL17 and SL18. This indicates that the spiral and welded wire fabric reinforcement effectively confined the cracked grout pocket region under large deformations, enabling significant force redistribution to develop. Hence, the confined specimens sustained large loads under significant deformations.

A direct comparison of the behavior for SL17 and SL18 demonstrated some similarities in overall behavior and the same capacity, although the spiral reinforcement confined the pocket more effectively at smaller deformations. Figure 3.60c shows the SL17 pocket and surrounding concrete confined within the spiral. The failure surface extended from the bar heads. Figure 3.60b shows a similar spalling above the top longitudinal reinforcement for both specimens, although the failure surface for SL18 did not clearly appear.

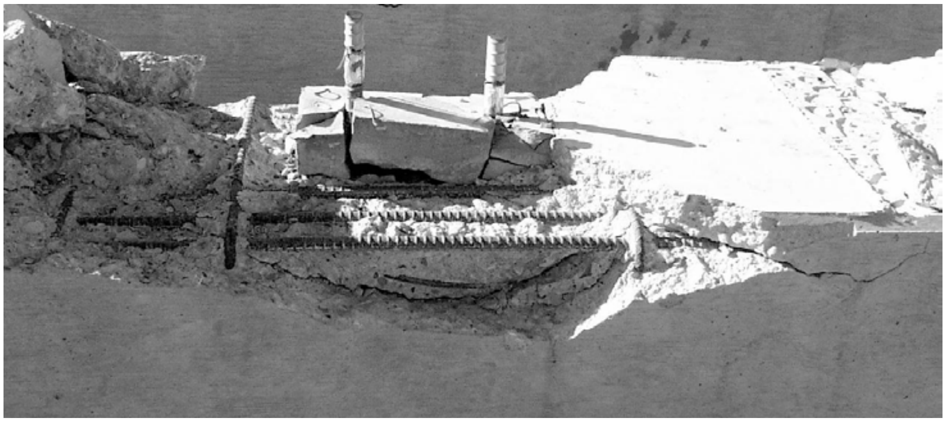
A marked decrease in stiffness was evident for SL18 at 25 kips, as corner cracks and splitting cracks formed simultaneously in the concrete. Although similar cracks formed at the same load for SL17, the spirally-confined specimen exhibited smaller crack widths and even exhibited crack discontinuity at the surface. The load-slip behavior and distribution of bar forces also indicated better confinement for SL17. Figure 3.59 shows a much softer load-slip response for SL17, corresponding to greater bond deterioration.



a. Spiral *b. Welded Wire Fabric*



c. Failure Surfaces for Confined Specimens



d. Failure Surface for Spirally-Confined Specimen

Figure 3.60 Failure Surfaces for Specimens Using Spiral and Welded Wire Fabric Confinement (SL17 and SL18)

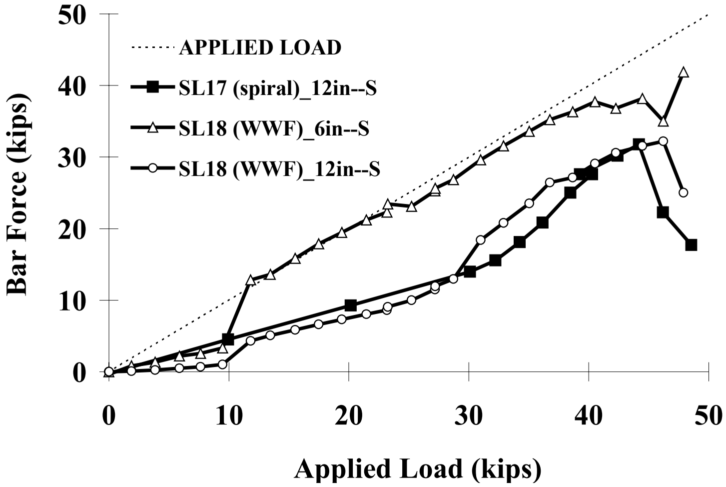


Figure 3.61 Distribution of Bar Forces for Confined Specimens (SL17 and SL18)

This implies that much larger grout pocket deformations were required to mobilize the welded wire fabric cage. Due to its rectangular shape, the welded wire fabric cage would require significant deformation before it could take on a shape more suitable for confinement. The more gradual unloading for SL17 can be seen in Figure 3.59, although the SL18 south bar (not shown) sustained load better than the north bar.

3.3.4.8 Bar Size

Two differences in behavior between #6 and #8 bars were observed for single-line grout pockets. As mentioned earlier, due to the shallow embedment requirements for the smaller #6 bars, yield was achieved in just 6 in. ($8d_b$). Because of the shallow embedment, the effects of splitting cracks were sometimes more pronounced. Figure 3.62 shows the total loss of bond for the headed #6 bar with a 4-in. embedment (SL09), and decreasing bond degradation as the embedment depth increased. Tests for #8 bars used larger embedment depths that provided a greater length for development of bond and, therefore, portrayed less severe splitting effects. In addition, the shallower embedment depths resulted in a greater amount of cracking in the grout pocket itself for some tests, such as SL09. Figure 3.50 shows that even when failure surfaces were more extensive, they still were much smaller than surfaces developed for #8 bars.

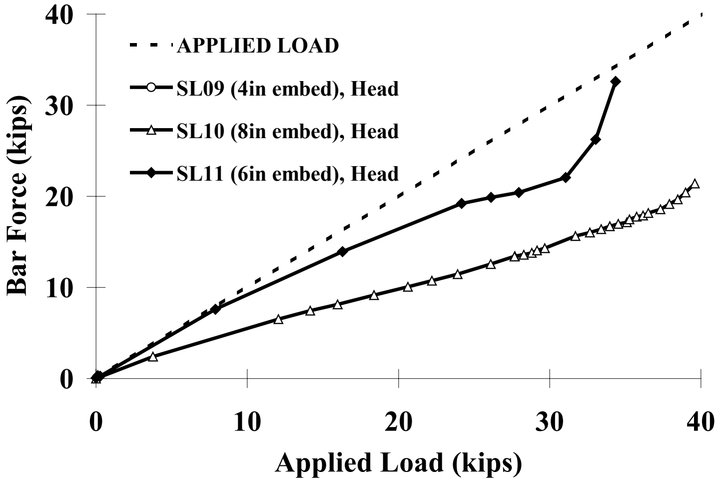


Figure 3.62 Distribution of Bar Forces for #6 Upset-headed Bars (SL09-SL11)

3.3.5 Conclusions

Based on pullout tests for epoxy-coated #8 and #6 bars grouted with MF928 grout in single-line grout pockets, the following conclusions can be made:

General—Single-line grout pockets are an acceptable alternative for precast bent cap connections and should be further investigated under the more realistic construction and loading conditions of Phases 2 and 3.

Failure Modes—Headed bars produced a concrete breakout failure, accompanied by significant cracking in the grout pocket and surrounding concrete. The breakout failure was sometimes preceded by bar yield. Straight bars failed by pullout accompanied by splitting cracks and a shallow cone.

General Behavior—Pullout behavior was characterized by the following stages of response to failure: Formation of splitting cracks in the grout pocket degraded bond, causing load transfer to the head and a reduction in connector stiffness. Development and growth of splitting cracks and grout pocket corner cracks in the concrete surrounding the grout pocket gradually reduced pocket confinement. Corner cracks combined with failure planes extending from the head to form a failure surface.

Embedment Depth—Typical bent cap sizes possess sufficient depth to develop epoxy-coated upset-headed and straight bars, even with a significant safety factor applied. Bar yield was achieved for epoxy-coated #8 bars at an embedment depth of $12d_b$ and at $8d_b$ for epoxy-coated upset-headed #6 bars (based on an average concrete compressive strength of 5.4 ksi). Confinement effects due to the concrete and longitudinal bars surrounding the grout pocket resulted in relatively small development lengths. To ensure ductile response, embedment depths should be selected so as to provide an adequate margin to account for connector yield strengths greater than 60 ksi and for strain hardening.

Grout Extension—*Masterflow 928 neat grout provided adequate strength and stiffness for application to a precast bent cap connection.* Nearly identical maximum loads were achieved for MF928 neat grout and MF928 grout extended with 3/8-in. pea gravel. Excellent interlock between the grout pocket and concrete interface developed for both cases, without surface roughening. Coarse aggregate in the extended grout matrix reduced the effects of splitting cracks on slip and enhanced stiffness. Neat grout, however, demonstrated adequate overall stiffness and strength for use in connections.

Grout Brand—*Masterflow 928 and Euclid Hi-Flow grouts provided acceptable force transfer characteristics.* Strength and stiffness compared closely for pockets grouted with MF928 and EHF, despite differences during grouting operations and more extensive grout pocket cracking for EHF.

Bar Anchorage—*Straight #8 bars developed anchorage sufficient for yield within depths typically available for precast bent caps.* Straight bar stiffness and strength compared closely to headed bar response. A #8 epoxy-coated straight bar achieved yield at an embedment depth of $12d_b$. Unlike headed bars, straight bars failed by bar pullout, accompanied by a shallow surface cone.

Multiple Bars—*Behavior of multiple bars loaded in a grout pocket connection was similar to single bar tests.* Multiple bars achieved approximately the same capacity as the corresponding single-bar specimen. The failure surface reflected more widely spread loading. (See Chapter 6 for a comparison of actual capacities to predicted capacities based on the Concrete Capacity Design Method.)

Grouted versus Cast-in-place Bars—*Grouted bars achieved a capacity 20% less than cast-in-place bars.* Grouted bars exhibited softer load-slip response and greater bond degradation, corresponding to a more disturbed tensile stress field. Reduced capacity and softer response are consistent with previously reported results for pullout behavior of anchors in cracked concrete and may be accounted for in design.

Confining Reinforcement—Spiral reinforcement and welded wire fabric are acceptable confining reinforcement alternatives for precast bent cap connections. Both spiral and welded wire fabric confining reinforcement increased capacity 50% over the unconfined specimen and increased lead displacements by a factor of at least 2.5 at the maximum load. Spirally-confined grout pockets limited crack growth and crack widths better than welded wire fabric, and exhibited a stiffer load-slip response with less severe bond degradation up to the maximum load. Welded wire fabric required significantly larger deformations before becoming fully effective. Constructability considerations for spiral reinforcement and welded wire fabric are discussed in Chapters 4 and 5.

Bar Size—A minimum embedment depth of 12 in. should be used for precast bent cap connections using #6 upset-headed bars to limit effects of splitting cracks. Shallow embedment depths used in testing of #6 bars led to loss of bond or more severe degradation of bond, even though bar yield was achieved at 6 in.

See Section 3.2.3.3 for lessons learned regarding grout performance and grouting operations. Test results are used in Chapter 6 to help develop a design methodology.

3.4 DOUBLE-LINE GROUT POCKET TESTS

3.4.1 Scope of Testing

Six double-line grout pocket specimens, DL01-DL06, were tested, each using two #6 epoxy-coated bars per pocket and MF928 neat grout. Table 3.7 shows the test matrix for this series of tests. The first four tests used upset-headed bars embedded 6 inches ($8d_b$). DL01 and DL02 compared response for unconfined specimens with bar configurations corresponding to transverse moment (one bar loaded in each pocket) and longitudinal moment (two bars loaded in one pocket), respectively. Based on a lower capacity that resulted for the longitudinal moment configuration, subsequent specimens used the longitudinal configuration. Similar to confinement used in single-line tests, DL03 and DL04 included spirals and welded wire fabric for confinement (Table 3.1) and, therefore, compare with DL02. The last two tests compared specimens that were confined with spirals and welded wire fabric but used straight bars embedded 9 inches ($12d_b$). Because small increases in capacity and ductility were achieved for the shallowly embedded bars ($8d_b$) used in DL03 and DL04, a deeper embedment was selected to better mobilize the confining reinforcement in DL05 and DL06. Straight bars were used to determine if the exceptional anchorage achieved by straight bars in single-line pocket tests would develop.

3.4.2 Summary of Results

Table 3.7 lists the test matrix and select test results for double-line grout pocket tests. Headed bars produced a concrete breakout failure, accompanied by significant cracking in the grout pocket and surrounding concrete. Straight bars failed by pullout accompanied by splitting cracks and a shallow cone, although the use of confining reinforcement produced some differences from single line grout pocket tests that used straight bars.

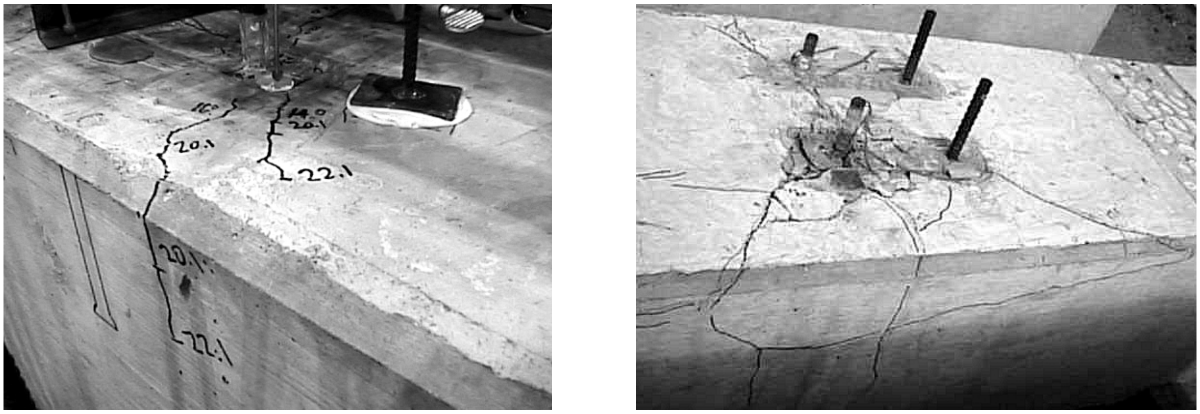
Table 3.7 shows that most grout cube and concrete cylinder strengths were fairly uniform, although DL01 had a grout strength considerably lower than the concrete strength due to the 10% water added for flowability during grouting. All tests exhibited response quite similar to single-line grout pocket behavior, and each test ended with a concrete breakout failure. To some extent, the DL05 and DL06 straight bar response was characteristic of single-line grout pocket pullout failures, although the presence of confining reinforcement enhanced ductility and spread cracking in the specimen. DL06 was the only test in which the bar yielded before failure. Load-slip curves demonstrated stiff, nearly linear response prior to significant cracking, followed by highly non-linear response as the maximum load was approached. Compared to single-line grout pocket tests, smaller splitting loads and crack widths resulted.

3.4.3 Effects of Variables

3.4.3.1 Bar Configuration

DL01 and DL02 response was very similar to that observed for unconfined single-line grout pocket specimens: splitting cracks formed across the grout pocket, splitting extended into the concrete, corner cracks developed, and eventually a concrete breakout failure occurred. However, the different positions of the loaded bars for transverse and longitudinal moments produced a slightly different failure surface that resulted in a capacity 20% larger for the transverse moment case (DL01).

Figures 3.63 and 3.64 compare the crack patterns for DL01 and DL02, respectively, as the specimens were loaded to failure. DL01 had a single bar loaded in each pocket. Because each loaded bar was positioned in the center of its respective pocket, splitting and corner cracks first formed adjacent to the loaded bar in each pocket, as shown in Figure 3.63a. The concrete breakout surface appeared on the side face closest to the loaded pocket (Figure 3.63b), as the bar was displaced beyond the maximum load. Data records indicate similar behavior occurred on the opposite side, although the failure surface did not extend to the surface of the beam. At failure, cracks appeared at both ends of the pocket. However, corner cracks developed only at the outside edge of pockets. Since the resultant of the total applied load was located at the center of the beam, a tensile stress distribution symmetric with respect to the beam centerline developed. This was similar to single-line grout pocket tests, as shown in the stress field idealization of Figure 3.65a.



a. Splitting and Corner Cracks *b. Failure Surface*

Figure 3.63 Cracks Patterns for Transverse Moment Specimen (DL01)

In contrast, DL02 loaded both bars in one pocket. Similar to single-line grout pocket tests, splitting cracks and corner pocket cracks developed, as shown in Figure 3.64a. However, a more pronounced surface spalling at each end of the grout pocket occurred due to the load location, the 7-degree taper along the beam, and the shallow embedment depth. Spalling normally occurred as the connector was displaced after the maximum load was reached (Figure 3.64b). Figure 3.64b also shows numerous horizontal splitting cracks, within the grout pocket, that extended from the bar. Figure 3.65b shows a schematic of the tensile stress distribution for DL02, based on the formation of cracks at failure. A comparison of Figures 3.65a and 3.65b, together with 3.63b and 3.64a show that the symmetrical loading in DL01 engaged a larger portion of the beam in resisting pullout, thereby leading to a higher capacity. In both DL01 and DL02, the presence of unloaded (dummy) bars and grout pockets did not appear to influence connector response.

Figure 3.66 indicates a similar load-slip response for DL01 and DL02, although a lower peak load and more sudden loss of load occurred for the longitudinal moment case, related to the smaller failure surface and greater proximity of the connectors to the beam side face. However, bond was maintained in a very similar manner for both cases, as shown in Figure 3.67, with only minor splitting effects apparent. At failure, connectors carried approximately 60% of the load at the head, corresponding to test results for shallowly embedded connectors in single-line grout pockets.

Table 3.7 Double-Line Grout Pocket Test Matrix and Select Results

						Materials			Load-Deflection			Cracking			
Test		Bars			hef	Grout	f'c (ksi)		Pyield^{D}	Pmax^{D}	Slip^{A}	Psplit^{D}	width	Pserv^{D}	width
ID	type	no.-size	anchor	confmt.	in.	brand	grout^{C}	concr	kips	kips	10^{-3} in.	kips	10^{-3} in.	kips	10^{-3} in.
DL01	DLGP-TM	2-#6	upset head	n/a	6	MF928	4.2	5.0	*	24	17	14	2	22	2
DL02	DLGP-LM	2-#6	upset head	n/a	6	MF928	5.9	5.1	*	20	<36^{B}	14	2	18	3
DL03	DLGP-LM	2-#6	upset head	spiral	6	MF928	5.8	5.6	*	22	<44^{B}	14	2	20	3
DL04	DLGP-LM	2-#6	upset head	wwf	6	MF928	5.4	5.6	*	24	74	12	2	-	-
DL05	DLGP-LM	2-#6	straight	spiral	9	MF928	6.3	5.7	*	24	67	8	5	22	7
DL06	DLGP-LM	2-#6	straight	wwf	9	MF928	6.4	5.8	27	28	55	6	2	24	5

Footnotes

A. Slip corresponds to head slip at maximum load, except as noted

B. Based on lead deflection at Pmax; head deflection unreliable

C. Grout cube strength modified by 0.8 factor

D. Load shown is the applied load per bar

Abbreviations

*=Concrete breakout before bar yield

MF928=Masterflow 928, neat

3.4.3.2 Confining Reinforcement

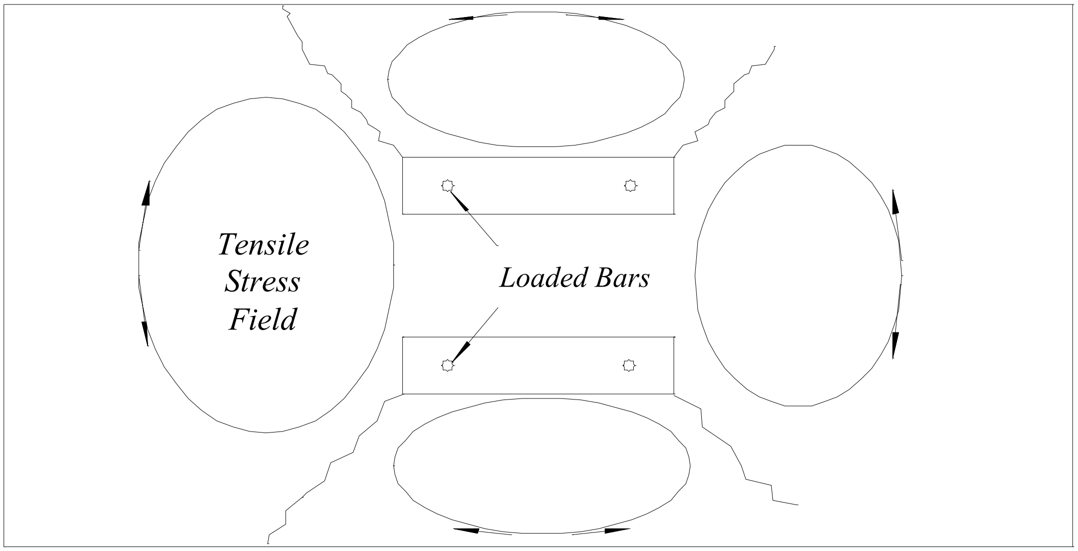
Using a longitudinal moment configuration, DL03 and DL04 compared behavior between specimens confined with spirals or welded wire fabric and the unconfined DL02 specimen. Figure 3.68 shows the crack patterns that developed at failure. Confining reinforcement surrounded both pockets, spreading the load effects and enabling cracks to extend to the far side of the beam. However, the overall crack pattern for confined specimens closely matched that for DL02 (compare Figures 3.64a and 3.68). Other similarities in response were observed, including similar stiffness up to yield (Figure 3.69) and the same rate of bond degradation (Figure 3.70) with the head taking most of the load at failure.

The load-slip plot in Figure 3.69 demonstrates that the spirally-confined grout pocket specimen achieved only a minor 10% increase in capacity and 75% increase in maximum lead displacement (at the load at which DL02 reached its maximum lead slip). The specimen confined with welded wire fabric performed slightly better, exhibiting a 20% increase in capacity and a 280% increase in maximum lead displacement. Similar to unconfined specimen behavior, DL03 and DL04 experienced only small slip at the peak load. Hence, the increase in capacity may be attributed to the intersection of cracks with the confining reinforcement, rather than to significant confinement forces. Spiral confinement in DL03 produced a notably smaller increase in capacity and ductility than in single-line specimens confined with spirals. Additionally, although the use of welded wire fabric produced a more ductile response, a significant loss of load occurred after just two load increments beyond the peak load.

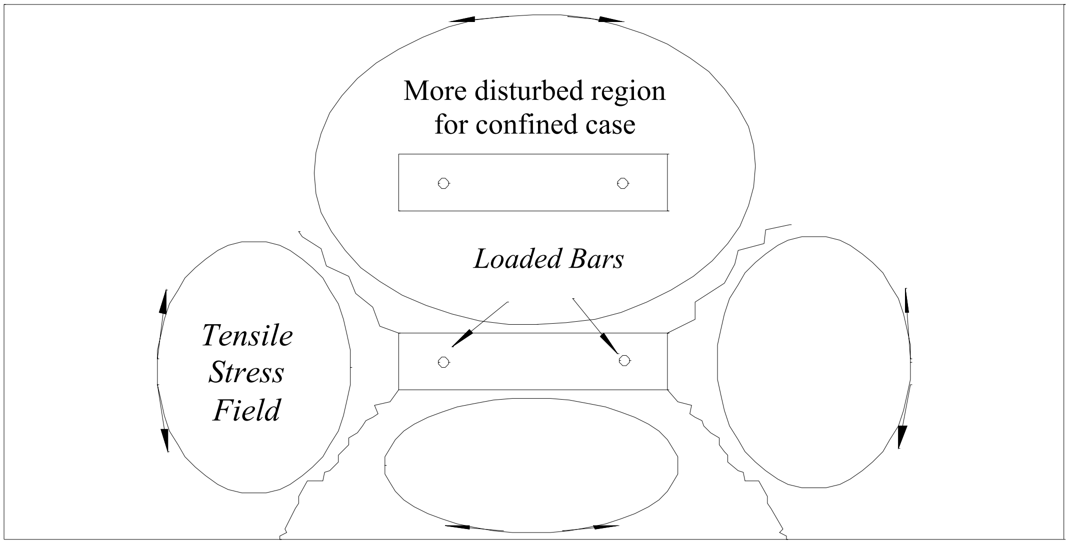
These characteristics indicate limited confinement effects. This is attributed to the use of a shallow 6-in. embedment depth for #6 bars and the placement of confining reinforcement beneath the longitudinal bars (Figure 3.11). Figure 3.71a illustrates the minimal intersection of an idealized failure surface with welded wire fabric. Figure 3.71b confirms that the majority of the failure surface along the beam was indeed above the DL04 confining reinforcement.

a. Cracks at Maximum Load *b. Exposed Surface Spalling*

Figure 3.64 Cracks Patterns for Longitudinal Moment Specimen (DL02)



a. Transverse Moment Configuration



b. Longitudinal Moment Configuration

Figure 3.65 Comparison of Tensile Stress Fields for Transverse and Longitudinal Moment Cases (DL01 and DL02); (compression struts not shown)

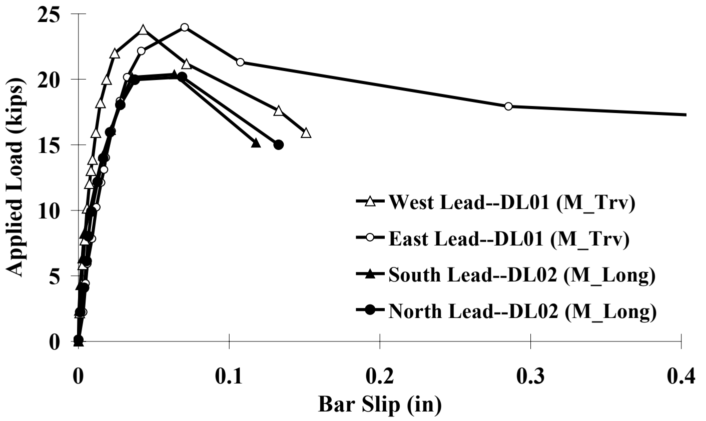


Figure 3.66 Load-Slip Behavior for Transverse and Longitudinal Moment Cases (DL01 and DL02)

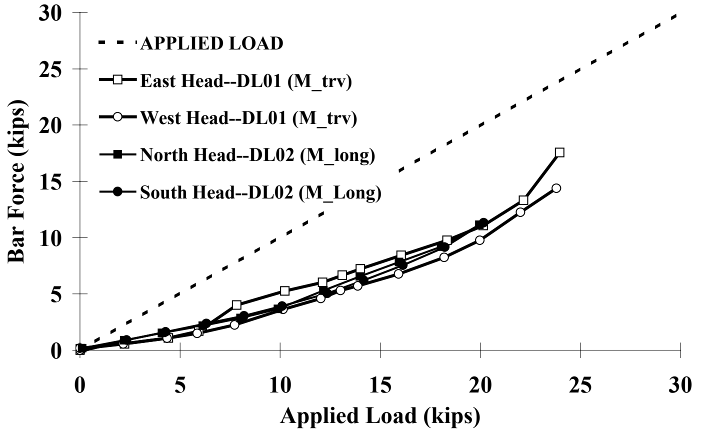
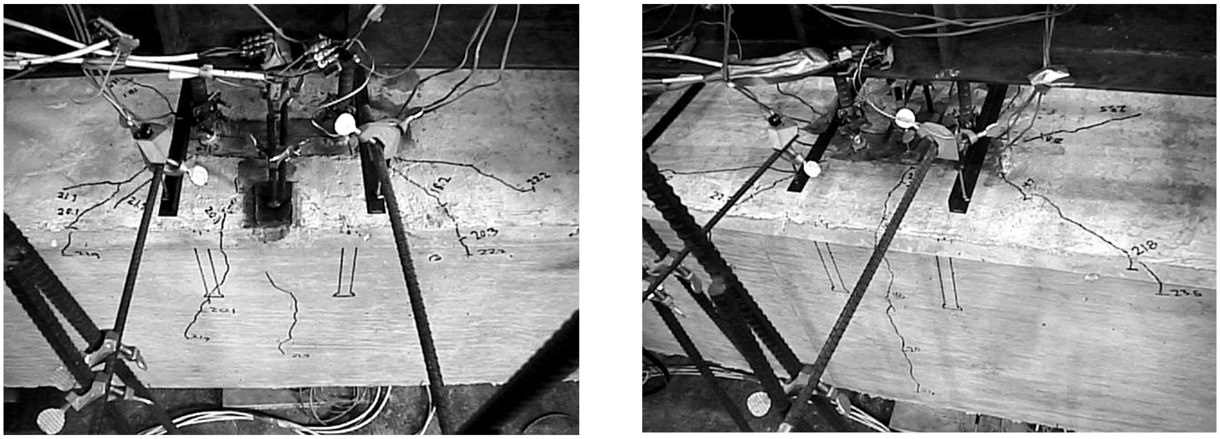


Figure 3.67 Bar Force Distribution for Transverse and Longitudinal Moment Cases (DL01 and DL02)



a. Spiral Confinement (DL03)

b. Welded Wire Fabric Confinement (DL04)

Figure 3.68 Crack Patterns at Maximum Load for Confined Specimens

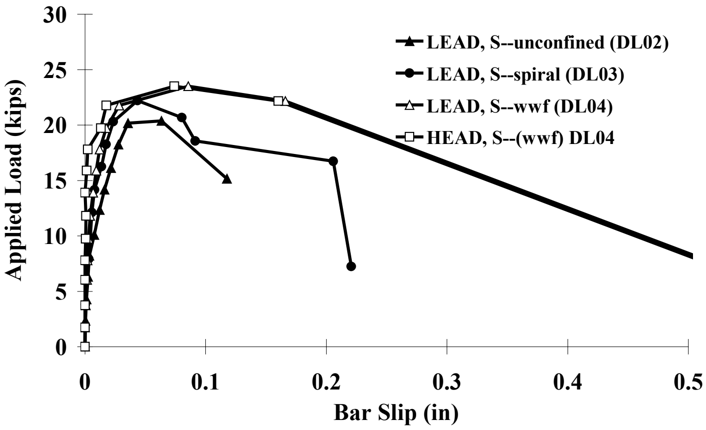


Figure 3.69 Load-Slip Behavior for Unconfined and Confined Specimens (DL02 and DL03-DL04)

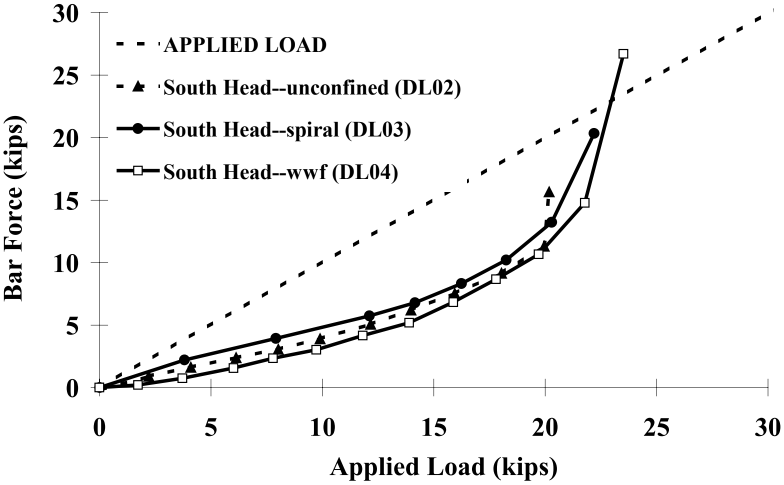


Figure 3.70 Bar Force Distribution for Unconfined and Confined Specimens (DL02 and DL03-DL04)

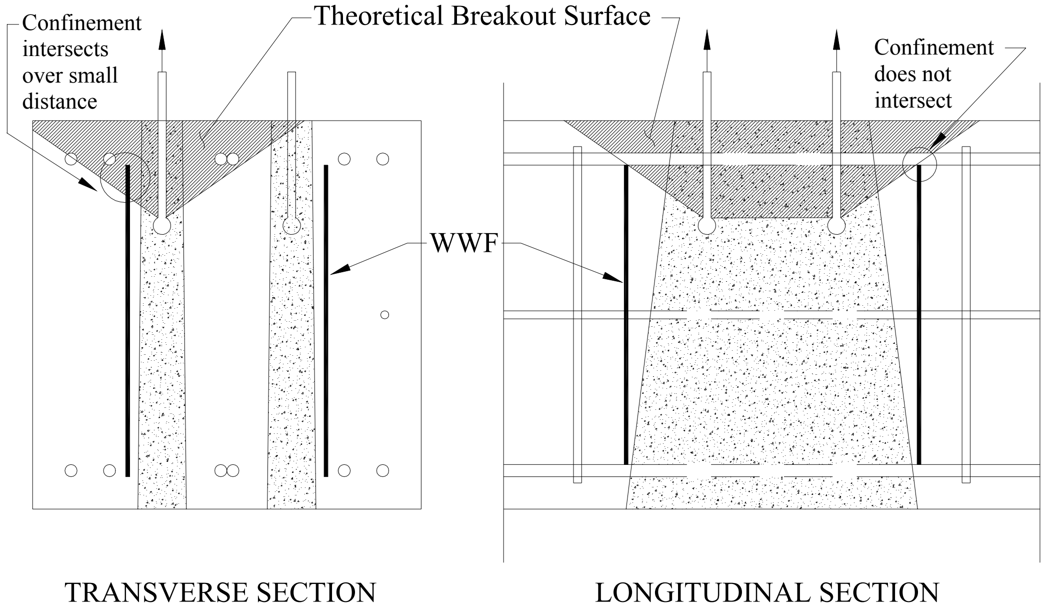
3.4.3.3 Bar Anchorage and Confinement

Using straight bars embedded 9 in. ($12d_b$), DL05 and DL06 compared specimens confined with spirals and welded wire fabric, respectively. A deeper embedment was intentionally chosen to better engage the confining reinforcement. To some extent, the DL05 and DL06 straight bar response was characteristic of single-line grout pocket pullout failures, although the presence of confining reinforcement enhanced ductility and spread cracking in the specimen. The spirally-confined DL05 achieved a maximum load of 24 kips, approximately 15% larger than the capacity for DL03, which used headed bars embedded 6 in.

DL06 achieved a load approximately 15% larger than DL05, just reaching bar yield prior to failure. Thus, straight epoxy-coated #6 bars achieved yield in $12d_b$, or 2/3 of that predicted for straight bar anchorage in concrete. The larger capacity in DL06 may indicate a greater intersection of cracks with the welded wire fabric mesh compared to the spiral in DL05.

Figure 3.72 compares the crack patterns at the maximum load. The nearly horizontal failure plane shown in Figure 3.72a developed at the last load increment. This shows the ability of the spiral reinforcement to cause the confined pocket to act to some extent as a whole. When a further displacement was imposed on the DL05 specimen after the peak load was reached, a shallow cone developed, similar to that in straight bar tests for single-line grout pockets. Figure 3.72b shows that only radial splitting cracks appeared on the surface for DL06, followed by shallow surface spalling at one end of the grout pocket after the peak load was reached. This is characteristic of a straight bar pullout failure.

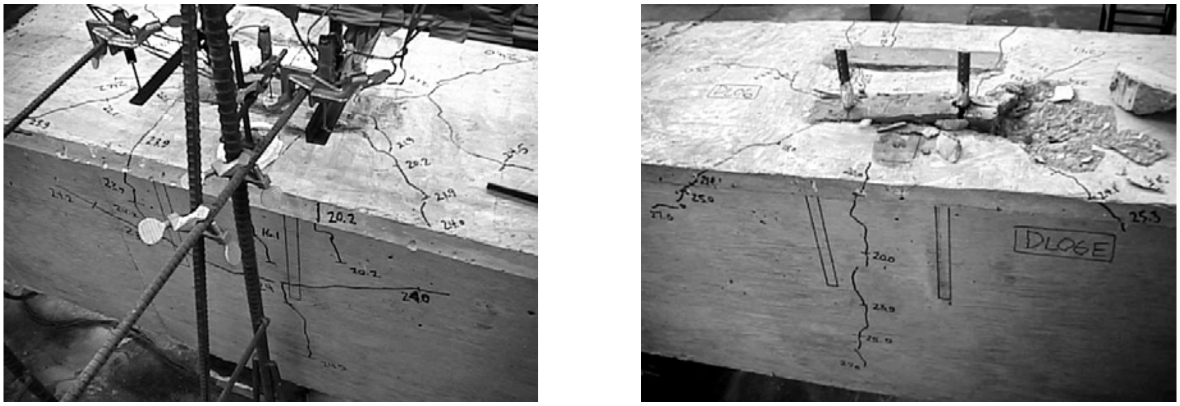
Load-slip behavior for the two cases is compared in Figure 3.73. The steep post-peak gradient for DL06 indicates rather ineffective confinement by the welded wire fabric. Thus, the larger capacity for DL06 compared to DL05 cannot be attributed to greater confinement forces. It should be noted that equal capacity was achieved for spiral and welded wire fabric confinement in single-line tests, which used the larger bar sizes and deeper embedment depths.



a. Intersection Between Theoretical Breakout Surface and Confinement [3.1]

b. Spalling above DL04 Confinement

Figure 3.71 Ineffective Confinement for Shallow Embedment Depth



a. Spiral Confinement (DL05) *b. Welded Wire Fabric Confinement (DL06)*

Figure 3.72 Crack Patterns at Maximum Load for Confined Specimens

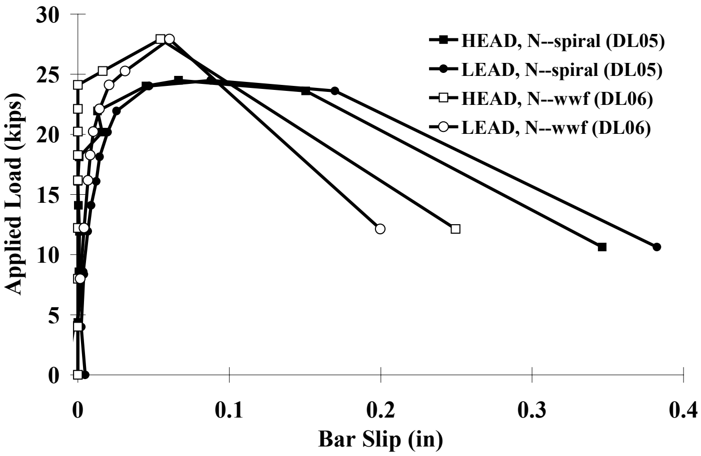


Figure 3.73 Load-Slip Behavior for Straight Bars in Confined Grout Pockets (DL05 and DL06)

DL05 portrays a greater ductility, with a displacement ductility ratio of approximately 2.5. Based on a comparison with DL03 (Figure 3.69), the increase in ductility is due at least in part to the effectiveness of confinement at a deeper embedment depth. The fact that headed bars confined with welded wire fabric in DL04 sustained load better than the more deeply embedded straight bars of DL06 is attributed to the difference failure mode for the two cases.

3.4.4 Conclusions

Based on pullout tests for #6 epoxy-coated bars grouted with MF928 grout in double-line grout pockets, the following conclusions can be made:

1. General—Double-line grout pockets are an acceptable alternative for precast bent cap connections and should be further investigated under the more realistic construction and loading conditions of Phases 2 and 3.
2. Failure Modes and General Behavior—Headed bars produced a concrete breakout failure, accompanied by significant cracking in the grout pocket and surrounding concrete. Straight bars exhibited a pullout failure, although spiral confinement caused the entire pocket to act to some extent as a whole.

3. Bar Configuration—*Design of a precast bent cap connection should account for both transverse and longitudinal moments.* The specimen loaded in a transverse moment configuration was subjected to symmetrical loading, which engaged a larger portion of the beam in resisting pullout. This resulted in a capacity 20% larger than the specimen loaded in the longitudinal moment configuration. Design methods can account for different failure surfaces.
4. Confining Reinforcement—Spiral rebar and welded wire fabric are acceptable confining reinforcement alternatives for precast bent cap connections. A minimum embedment depth of 12 in. is recommended to ensure confining reinforcement is engaged. Due to the use of a shallow embedment depth ($8d_b$) and placement of confining reinforcement beneath longitudinal bars in the cap beam, the use of spirals and welded wire fabric to confine two upset-headed #6 bars provided a small 10-20% increase in capacity over the unconfined specimen. Lead displacements increased by a factor of 1.7 and 2.8, respectively, although the peak load was not well sustained. Spirally-confined straight #6 bars exhibited a displacement ductility ratio of 2.5, but bars confined by welded wire fabric exhibited a steep post-peak gradient, indicating a lack of confinement for shallow embedment depths.
5. Bar Anchorage—Straight #6 bars developed anchorage sufficient for yield within depths typically available for precast bent caps. Straight #6 bars achieved yield in $12d_b$ (based on an average concrete compressive strength of 5.4 ksi). This was due to confinement effects of the surrounding concrete, as well as use of confining reinforcement.

3.5 GROUTED VERTICAL DUCT TESTS

3.5.1 Scope of Testing

Eight grouted vertical duct specimens, VD01-VD08, were tested using a #11 epoxy-coated bar grouted into a 4-in. diameter steel corrugated duct with MF928 neat grout. Table 3.8 shows the test matrix, which reflects a re-ordering of original test ID's to simplify discussion of results. The variables selected for testing were chosen to maximize comparisons for the limited number of tests. The primary variables included embedment depth, grout brand, and bar anchorage.

The effect of embedment depth on behavior was compared for two different grout brands, MF928 (VD01, VD03, VD07) and EHF (VD04, VD08). Because confinement effects were expected to enhance anchorage, straight bars were tested at embedment depths of 12, 18, and 24 in. (from $8.5d_b$ to $17d_b$), which are much smaller than the 32 in. ($23d_b$) expected on average for straight bar anchorage in concrete. After bars were found to reach yield at an 18-in. embedment, a shallower 12-in. embedment depth was used to determine the transition between bar yield and pullout. The 24-in. embedment was selected to investigate response for an embedment depth closer to what may be used in a precast bent cap.

Trial batches revealed differences in grout properties for different grout brands. Therefore, grouted vertical duct tests compared MF928, EHF, and Sika 212 in VD03-VD05, respectively, for straight bars embedded 18 in. Behavior for MF928 and EHF was also compared at a 24-in. embedment depth in VD07 and VD08. In addition, straight bar response was compared to that for upset-headed bars at embedment depths of 12 in. (VD01, VD02) and 18 in. (VD03, VD06). MF928 was used for these tests.

3.5.2 Summary of Results

Table 3.8 shows the test matrix and select test results for grouted vertical duct tests. Headed and straight bars using an embedment depth of 12 in. exhibited pullout failures, characterized by development of splitting cracks in the surrounding concrete and pullout of the bar-grout mass from the duct. All but one of the bars using an embedment depth greater than 12 in. were loaded to the onset of pullout failure and exhibited similar response up to the maximum load. However, these specimens were unloaded prior to complete pullout of the bar-grout mass.

Table 3.8 Grouted Vertical Duct Test Matrix and Select Results

					Materials			**Load-Deflection**			**Cracking**			
Test		**Bars**		**hef**	**Grout**	**f'c (ksi)**		**Pyield**	**Pmax**	**Slip**[C]	**Psplit**	**width**	**Pserv**	**width**
ID	**type**	**no.-size**	**anchor**	**in.**	**brand**	**grout**[D]	**concr**	**kips**	**kips/bar**	**10^{-3} in.**	**kips**	**10^{-3} in.**	**kips**	**10^{-3} in.**
VD01	GVD	1-#11	straight	12	MF928	4.2	5.4	*	76	<56[B]	28	2	48	7
VD02	GVD	1-#11	upset head	12	MF928	4.2	5.4	*	92	85	32	2	80	3
VD03	GVD	1-#11	straight	18	MF928	5.7	5.6	92	119[A]	29, 93	28	2	80	5
VD04	GVD	1-#11	straight	18	EHF	3.1	5.6	*	94	93	12	2	60	7
VD05	GVD	1-#11	straight	18	SIKA	3.8	5.5	93	114[A]	31, 89	32	2	68	2
VD06	GVD	1-#11	upset head	18	MF928	4.8	5.5	93	120[A]	19, -	28	2	80	5
VD07	GVD	1-#11	straight	24	MF928	5.2	5.5	94	100[A]	<74[B],131[B]	32	2	72	2
VD08	GVD	1-#11	straight	24	EHF	4.5	5.5	93	118[A]	<57[B], -	20	2	84	2

Footnotes

A. Test stopped due to limitations of the test setup or to prevent bar fracture

B. Based on lead slip at Pmax; head slip unreliable

C. Slip corresponds to head slip at maximum load, except as noted. Slip includes beam displacement, due to unreliable beam records.

Actual slip for an 18-in. or 24-in. embedment depth is expected to be reduced by 0.02 in. at 94 kips, based on reliable VD04 record.

D. Grout cube strength modified by 0.8 factor

Abbreviations

GVD=Grouted Vertical Duct

*=Concrete breakout before bar yield

MF928=Masterflow 928, neat

EHF=Euclid Hi-Flow, neat

SIKA=Sika 212, neat

Table 3.8 shows that grout cube strength varied widely, between 3.1 and 5.7 ksi, and was less than the concrete compressive strength for seven of the eight tests. For some cases, the lower strength was due to the short time between grouting and testing, and for other cases it was related to the addition of water during remixing (VD04, VD08). Concrete strengths were nearly uniform at 5.5 ksi.

As shown in Table 3.8, three bars did not reach yield, but failed by pullout. Two of these tests used a shallow 12-in. ($8.5d_b$) embedment depth, and one had a low 3.1-ksi grout strength. The other five tests reached bar yield, but had the maximum load restricted by limitations of the test setup or possible bar fracture. Excellent bond was maintained between the cap and duct for all tests.

Load-slip curves demonstrated stiff response up to the peak load for the specimens that achieved bar yield. Other tests indicated a significantly lower stiffness. Overall, minor crack widths occurred at splitting and service level loads, although the specimen grouted with EHF (VD04) exhibited more cracking compared with specimens of similar embedment depth.

3.5.3 General Behavior

3.5.3.1 Overview

Pullout response for straight and headed bars grouted in a vertical duct was characterized initially by splitting cracks in the grout, followed by cracks in the surrounding concrete. Bond between the bar and grout was maintained within the duct when cracking in the surrounding concrete was limited. However, when splitting cracks in the concrete opened significantly (to ~0.005 to 0.009 in. widths) concrete confinement around the duct deteriorated, allowing the duct to dilate. This resulted in a much softer load-slip behavior and increased slip between the duct and grout, but did not prevent a subsequent increase in load. For embedment depths of 18 in. or more, most bars were loaded beyond the yield load and to the onset of pullout failure. However, loading was discontinued due to various limitations. Initial response was similar for embedment depths of 12 in. However, these bars exhibited pullout of the bar-grout mass from the duct prior to bar yield.

3.5.3.2 Illustration

The response of VD03 is described to demonstrate general behavior. VD03 used a single #11 straight bar grouted 18 in. into a vertical duct with MF928 neat grout. This particular test was selected because implementation of vertical duct connections in a precast bent cap system will normally use an embedment depth sufficient for bar yield and a grout strength comparable to the concrete strength. Important characteristics of a complete pullout failure are described in Section 3.5.4.2.

Splitting cracks in the grout between the bar and vertical duct were first observed at a load of 28 kips. The load-slip plot in Figure 3.74 shows a reduction in stiffness at the lead end corresponding to the formation of splitting cracks. Effects of splitting on bar force distribution were much less pronounced for grouted vertical ducts than for grout pockets, as shown in Figure 3.75. This indicates a less sudden loss of bond, as well as more effective confinement. Figure 3.76 shows multiple splitting cracks emanating radially into the surrounding concrete at loads between 56 kips and 72 kips. Such response is typically associated with pullout of concrete anchors. This indicates adequate bond developed at the interfaces between the bar, grout, duct, and concrete, enabling the bar and grouted mass to act as a unit. Figure 3.77 shows the gradual increase in duct strain with load, confirming that duct confinement was mobilized.

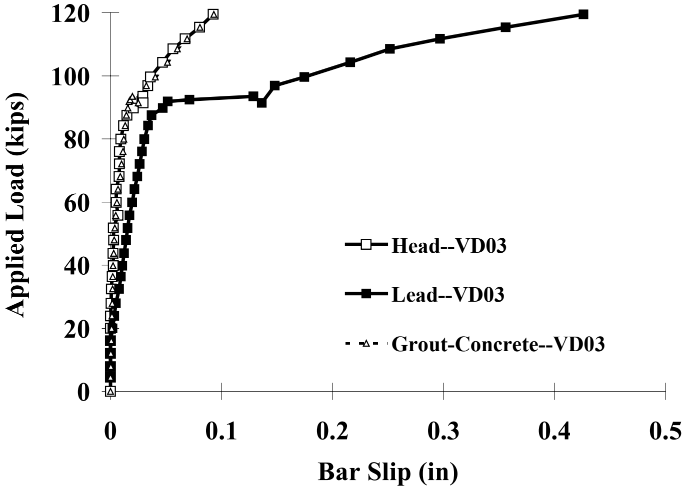


Figure 3.74 Load-Bar Slip Response (VD03)

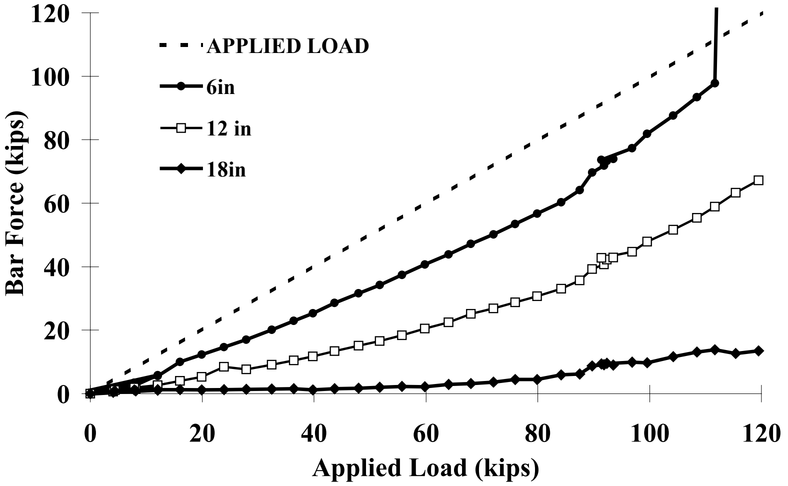


Figure 3.75 Distribution of Bar Forces (VD03)

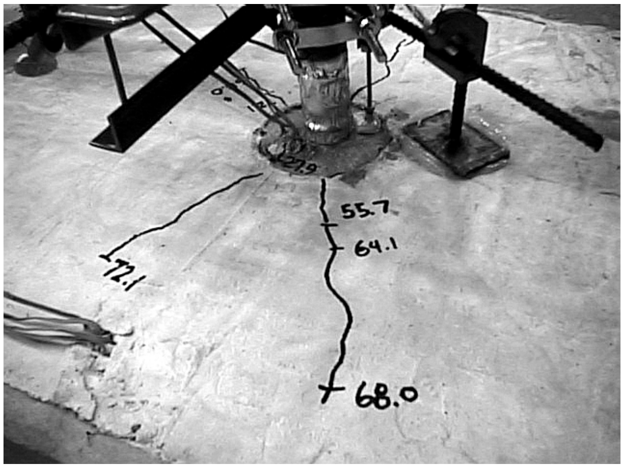


Figure 3.76 Spread of Splitting Cracks (VD03)

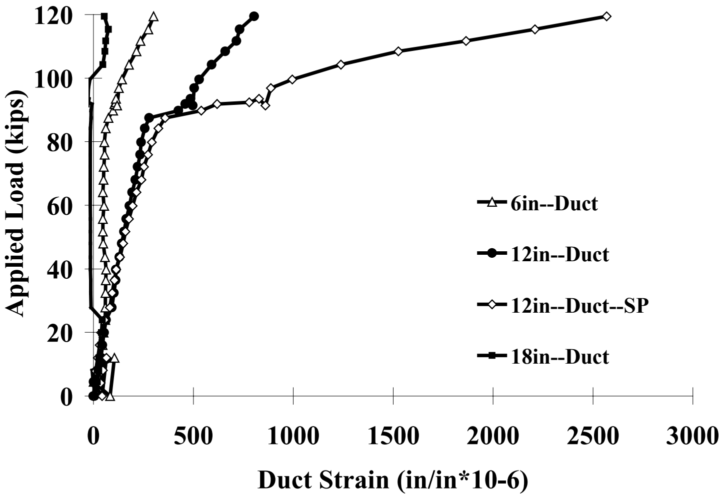


Figure 3.77 Load-Duct Strain Response (VD03)

As the applied load was increased, cracks spread further into the beam and down the side faces, as shown in Figure 3.78. At 88 kips, several new cracks appeared at the beam top surface, and existing cracks opened noticeably and extended down the side faces. This was matched by a sudden increase in duct strain at 12 in. (circumferential and spiral directions), indicating a loss of concrete confinement around the ducts. This led to dilation and yielding of the duct, as well as slip between the grout and duct. Slip was confirmed by the appearance of a crack around the entire periphery of the duct at the same load at which load-slip response at the grout-concrete interface suddenly softened (Figure 3.74). This indicates that the effectiveness of duct confinement of the bar depends on confinement of the surrounding concrete, as well as the integrity of the grout within the duct. It should be noted that the significantly larger strains in the spiral direction indicate the tendency of the duct to expand along the spirals.

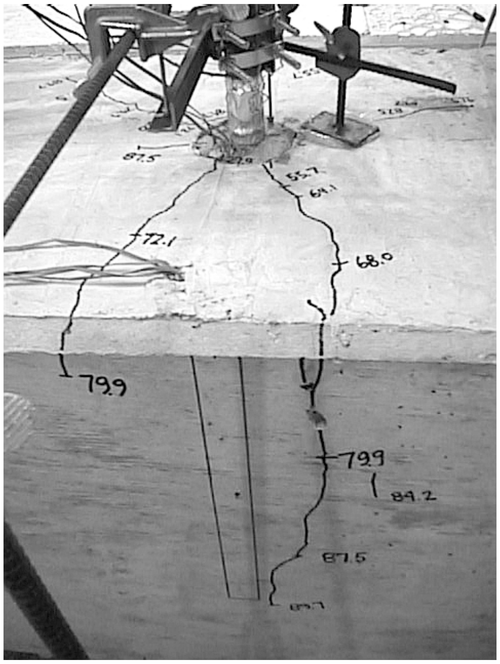


Figure 3.78 Crack Pattern at Bar Yield (VD03)

At 92 kips the bar yielded, as portrayed clearly in the lead slip record. Upon further loading, cracks extended and opened to widths as large as 0.025 in. Although excellent interlock between the duct and concrete was maintained, equal slip at the bar end and grout-concrete interface indicates pullout of the entire grout-bar mass as a rigid body within the duct. Diagonal cracks also appeared on the side faces, as shown in Figure 3.79. Figure 3.80 shows a close-up of the duct surface at maximum load. Loading was discontinued at 119 kips due to test setup limitations.

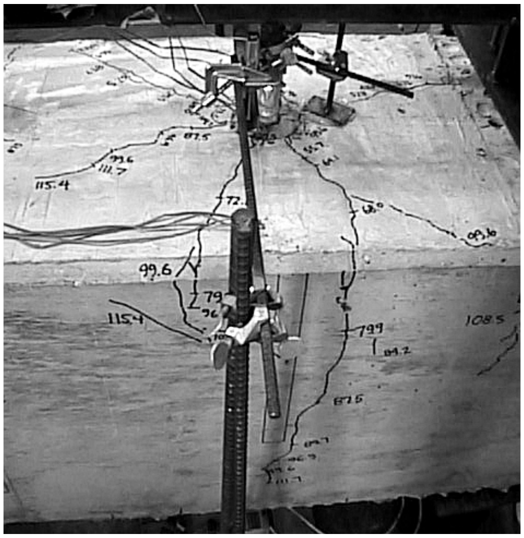


Figure 3.79 Crack Pattern at Maximum Load (VD03)

Figure 3.80 Cracking at Surface of Grouted Duct (VD03)

3.5.4 Effects of Variables

3.5.4.1 Embedment Depth

The embedment depth was varied to establish a transition between concrete pullout and bar yield. Table 3.8 shows that bar yield was achieved for straight and headed #11 bars at an embedment of 18 in. ($13d_b$). One exception was VD04, which had an unusually low grout strength of 3.1 ksi. Confinement effects of the duct and surrounding concrete enabled bars to reach yield at a relatively shallow embedment depth.

The effect of embedment depth on behavior is shown in Figure 3.81, which compares load-slip response for bars grouted 12 in., 18 in., and 24 in. with MF928 (VD01, VD03, and VD07, respectively). VD03 response described earlier demonstrated that stiffness at the bar end was directly related to the amount of beam cracking and subsequent loss of confinement around the duct. For VD07, the 24-in. embedment depth allowed the bar to engage a larger portion of the beam in resisting splitting. This resulted in less extensive cracking of the concrete and, hence, a much stiffer response. With just a 12-in. embedment, splitting cracks extended into the surrounding beam and opened at lower loads, producing a softer load-slip response. A similar load-slip relationship was observed for EHF with 18-in. and 24-in. embedment depths. Incorporation of sufficient confining reinforcement in the concrete around the ducts may enhance capacity and/or ductility.

Figure 3.82 compares bar force distribution for embedment depths of 18 and 24 in. (VD03, VD07). In contrast to the highly non-linear bar distributions for grout pocket specimens, a nearly linear relationship is portrayed at all depths, with bond forces between each 6-in. increment being approximately equal. For example, at an applied load of 80 kips, bond in the middle third of the bar (i.e., between 6 and 12 in.) resisted approximately 33% of the load for VD03, whereas approximately 22% was carried by bond between 6 and 12 inches for VD07. In addition, only a small non-linearity is evident as the maximum load is approached. This indicates well-distributed bond along the entire bar length and merely minor effects due to splitting and bond deterioration.

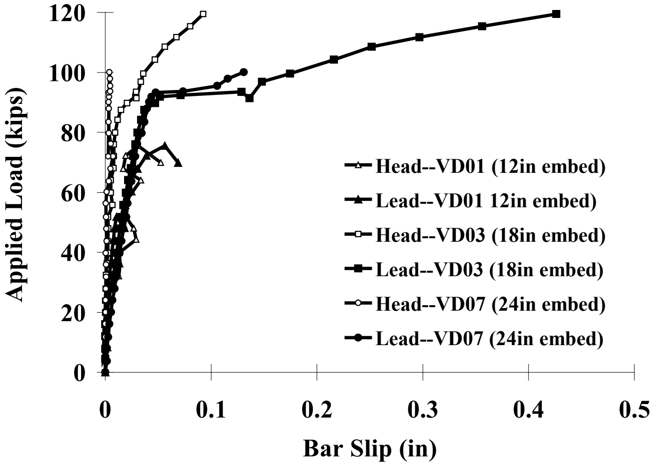


Figure 3.81 Load-Slip Behavior for Different Embedment Depths (MF928)

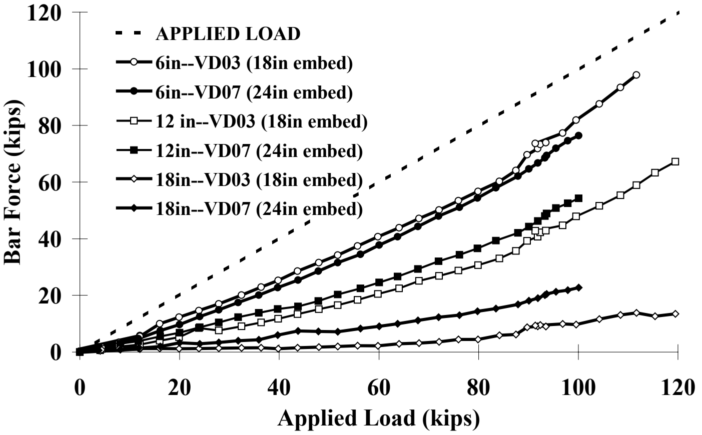


Figure 3.82 Bar Force Distribution for Different Embedment Depths (MF928)

3.5.4.2 Grout Brand

VD03-VD05 were designed to compare response of a straight #11 bar grouted 18 in. into a vertical duct using MF928, EHF, and Sika 212, respectively. The EHF grout used in VD04 required a 10% addition of mixing water to achieve adequate workability, which caused a significant reduction in grout strength. Test-day grout cube strengths for VD03-VD05 tests were 5.7 (12-day), 3.1 (10-day), and 3.8 ksi (9-day), respectively. Concrete strengths were all nearly 5.6 ksi. Therefore, this comparison of grout brands may also be considered a comparison of the effect of grout strengths.

Due to lower tensile strength, VD04 exhibited grout splitting at a much lower load than VD03 and VD05 (Table 3.8), as well as more extensive cracking. A softer load-slip response for EHF is evident (Figure 3.83), starting from the formation of beam splitting cracks at 50 kips. Splitting cracks in the beam opened to larger widths for VD04 due to the more extensive grout cracking and hence larger duct dilation. Beam splitting directly correlated to a reduction in confinement around the duct, as evidenced by spiral duct strains at the 12-in. depth shown in Figure 3.84. Loss of confinement eventually resulted in slip between the grout and duct at only 68 kips. This culminated in a pullout failure for VD04 (Figure 3.85). Slip was observed between the EHF grout and duct at a load of only 68 kips. The reversal of duct strain at the 6 in. depth for VD04 at loads greater than 84 kips indicates unloading due to pullout of a large grout mass near the surface.

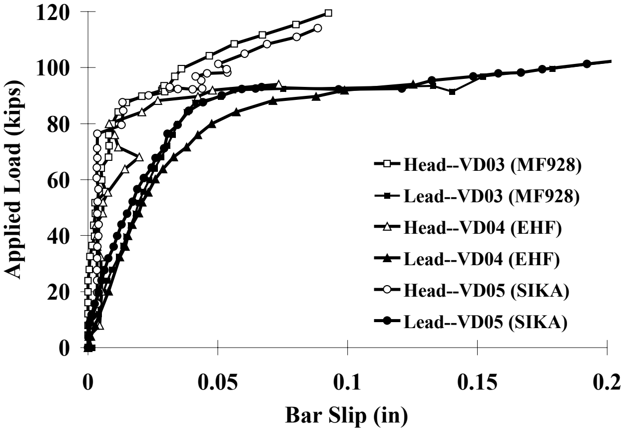


Figure 3.83 Load-Slip Behavior for Different Grout Brands (18-in. embedment)

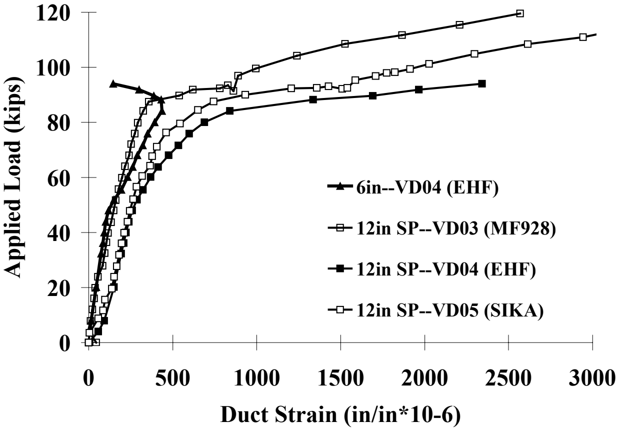


Figure 3.84 Load-Duct Strain Behavior for Different Grout Brands (18-in. embedment)

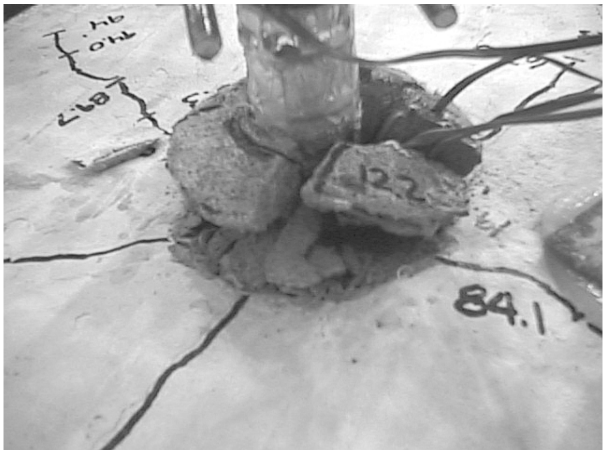


Figure 3.85 Pullout Failure (VD04)

Although ducts grouted with MF928 and Sika 212 exhibited some softening upon formation of beam splitting cracks, fairly stiff response continued until beam cracks widened just prior to failure. The stiffer response of VD03 compared to VD05 is attributed to the higher grout strength (5.7 vs. 3.8 ksi), which provided greater resistance to tensile splitting stresses and a larger modulus of elasticity. VD03 and VD05 bars reached yield and were loaded into the strain-hardening region before loading was discontinued. This comparison highlights the importance of using grout with adequate strength.

VD07 and VD08 also compared pullout response between MF928 and EHF, respectively, for bars embedded 24 in. In contrast to VD03 and VD04 comparisons, these tests portrayed similar response. Figure 3.86 shows a slightly softer load-slip behavior for EHF, which had a grout cube strength of 4.5 ksi, compared to 5.2 ksi for MF928. A primary difference in response was slip at the grout-duct interface. Very little slip for MF928 occurred at 100 kips, whereas EHF showed signs of pullout of the grout-bar mass.

Compared to VD04, splitting cracks in the EHF grout for VD08 occurred at a larger load, and much less grout cracking was observed during loading. The stiffer response and higher capacity for VD08 may be attributed to both the higher strength and deeper embedment.

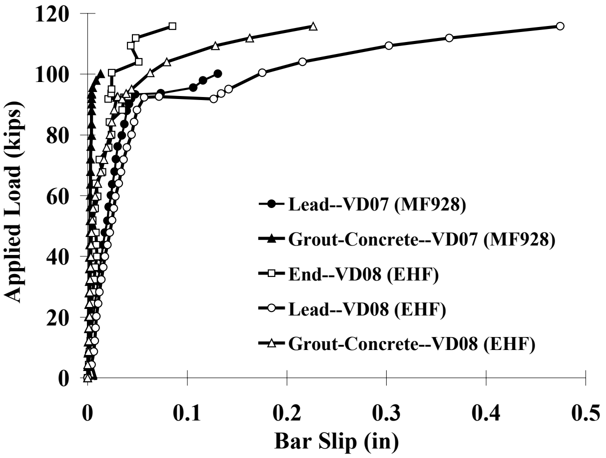


Figure 3.86 Load-Slip Behavior for Different Grout Brands (24-in. embedment)

3.5.4.3 Bar Anchorage

Two sets of tests were conducted to compare pullout behavior for straight and upset-headed bars grouted with MF928. VD01 and VD02 used an embedment depth of 12 in., and VD03 and VD06 used a depth of 18 in.

For the 12-in. embedment depth, both bars failed by pullout, with the headed bar achieving a capacity 20% larger than the straight bar. Data records portrayed considerable differences in response. Figure 3.87 shows the crack patterns for VD01 and VD02 at failure. Behavior for the straight bar exhibited features similar to those described in Section 3.5.3, although the failure surface indicated mobilization of a concrete breakout surface simultaneous to pullout failure.

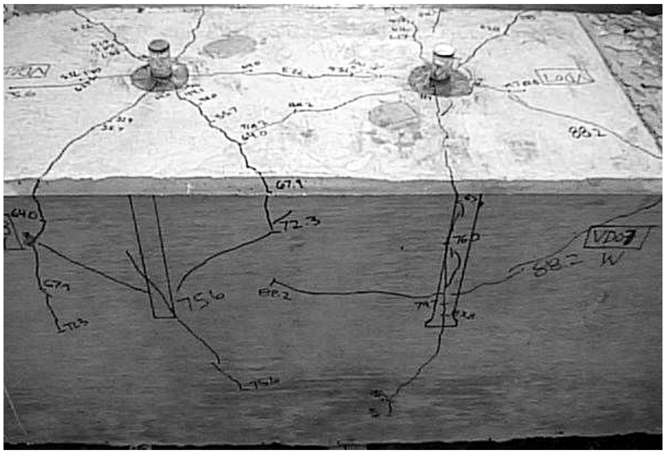


Figure 3.87 Crack Pattern at Failure for Straight and Headed Bars (12-in. embedment)

Figure 3.88 exhibits a much softer load-slip behavior at the lead end of the headed bar. For VD02, splitting initiated near the head at a small load of approximately 15 kips (Figure 3.89). This was similar to behavior observed in grout pocket tests using headed bars at shallow embedment depths. Rapid increase in VD02 duct strains starting at 40 kips indicates large local stresses around the head. However, splitting cracks did not appear at the beam surface until a load of 68 kips. Figure 3.90 shows the head carried 2/3 of the maximum load and therefore provided significant anchorage. As for straight bar tests, failure by pullout was initiated by loss of confinement associated with large splitting cracks in the beam. However, after the maximum load was reached, the use of an upset head allowed the lead slip to double with only a minimal loss of load. In addition, the large horizontal crack at failure may indicate more widespread effects for headed bars.

In contrast to behavior observed for a 12-in. embedment depth, VD03 and VD06 demonstrated similar response for an 18-in. embedment depth. Both headed and straight bars achieved bar yield, and exhibited similar load-slip response, as shown in Figure 3.91. Figure 3.92 indicates well-distributed bond forces, although, as expected, larger forces are clearly carried at the head for VD06. Only minor effects of splitting and small differences in duct strains were evident. The use of headed bars does provide connection redundancy, but test results indicate little advantage at deeper embedment depths for which bar yield is achieved.

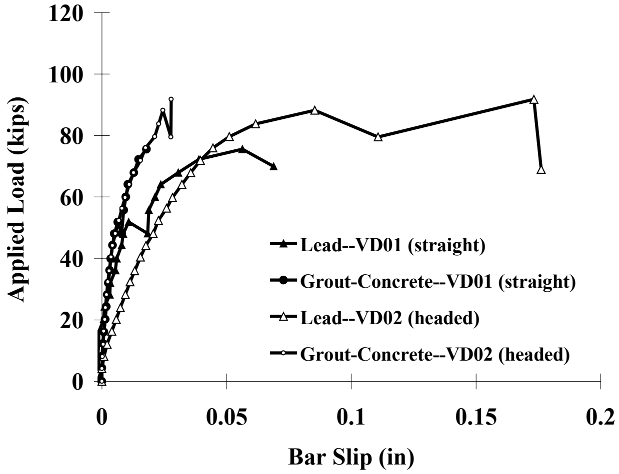


Figure 3.88 Load-Slip Behavior for Straight and Headed Bars (12-in. embedment)

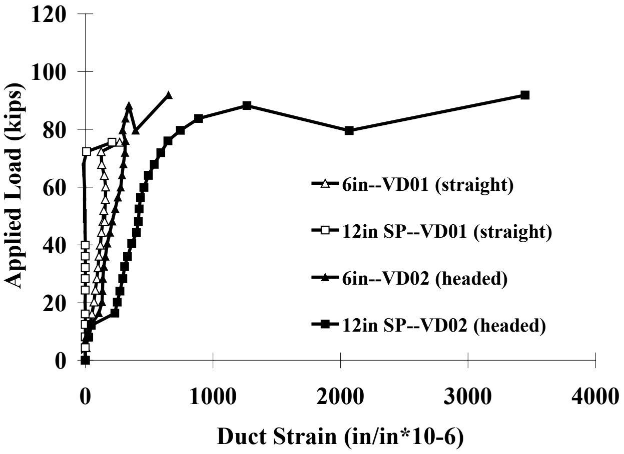


Figure 3.89 Load-Slip Behavior for Straight and Headed Bars (12-in. embedment)

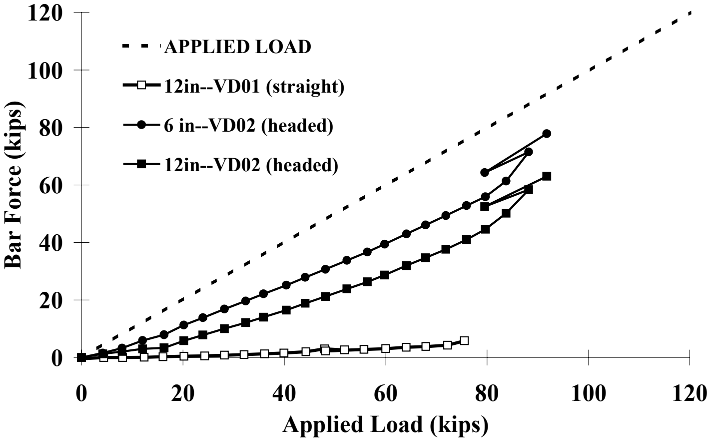


Figure 3.90 Bar Force Distribution for Straight and Headed Bars (12-in. embedment)

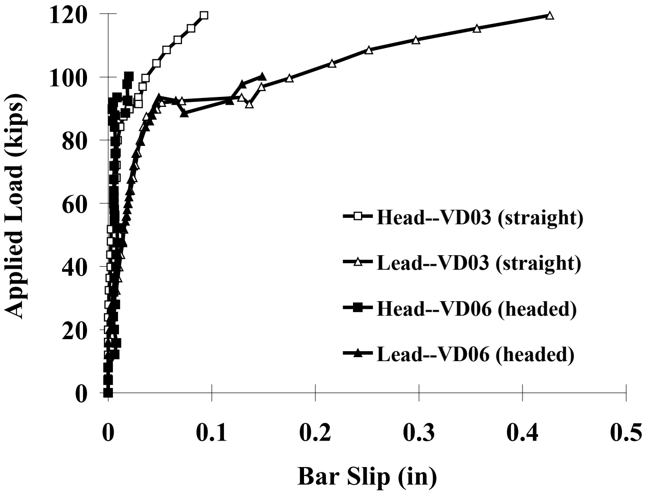


Figure 3.91 Load-Slip Behavior for Straight and Headed Bars (18-in. embedment)

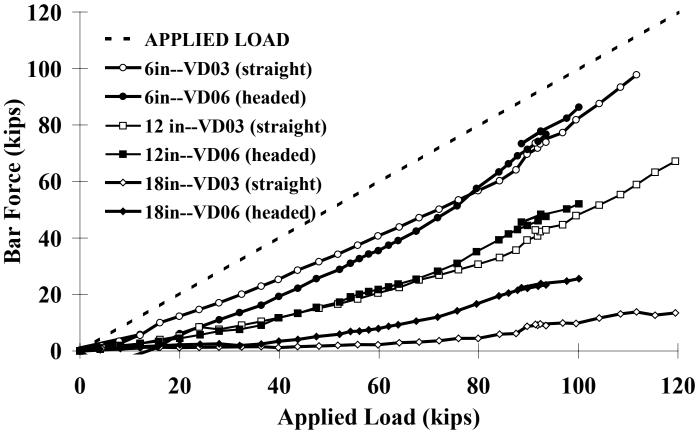


Figure 3.92 Bar Force Distribution for Straight and Headed Bars (18-in. embedment)

3.5.5 Conclusions

Based on pullout tests for epoxy-coated #11 bars grouted in 4-in. diameter steel corrugated ducts, the following conclusions can be made:

1. General—Grouted vertical ducts are an acceptable alternative for precast bent cap connections and should be further investigated under the more realistic construction and loading conditions of Phases 2 and 3.
2. Failure Mode—Headed and straight bars exhibited pullout failure, characterized by development of splitting cracks in the surrounding concrete and pullout of the bar-grout mass from the duct. All but one of the bars using an embedment depth greater than 12 in. were loaded to the onset of pullout failure and, up to the maximum load, exhibited response similar to bars that exhibited complete pullout of the bar-grout mass.

3. General Behavior—Pullout behavior was characterized by the following: Well-distributed bond developed along the bar, with only minor effects of grout splitting for most cases. Splitting cracks emanated radially from the duct into the surrounding concrete, indicating excellent bond between the bar, grout, duct, and concrete. Growth of splitting cracks in the beam reduced concrete confinement of the duct, leading to dilation of the duct and subsequent pullout of the grout-bar mass from the duct. Use of confining reinforcement around the ducts may increase ductility and capacity. Slip at the duct-concrete interface did not occur.

4. Embedment Depth—Typical bent cap sizes possess sufficient depth to develop epoxy-coated upset-headed and straight bars, even with a significant safety factor applied. Bar yield was achieved for epoxy-coated straight and upset-headed #11 bars at an embedment depth of $13d_b$ (based on an average concrete compressive strength of 5.5 ksi). Confinement effects due to the duct and surrounding concrete resulted in relatively small development lengths. Yield was nearly achieved for a headed bar at $8.5d_b$. To ensure ductile response, selection of embedment depths should also consider bar yield strengths greater than 60 ksi and strain hardening forces.

5. Grout Brand—Grout strengths differed significantly for three grout brands, leading to significantly different response for some cases. Twenty-eight day modified grout cube strength should be greater than the specified 28-day concrete compressive strength of the bent cap. Low grout strength for Euclid Hi-Flow led to more extensive grout cracking, softer load-slip behavior, lower capacity, and pullout failure before bar yield. Chapter 7 discusses a recommended margin between the grout cube strength and the compressive strength of the cap.

6. Bar Anchorage—Straight #11 bars developed anchorage sufficient for yield within depths typically available for precast bent caps. At an embedment depth sufficient for bar yield ($13d_b$), straight and headed bar response was very similar. The use of headed bars provides connection redundancy, but test results indicate little advantage at embedment depths required for bar yield. At a smaller embedment of $8.5d_b$, the headed bar achieved a capacity 20% larger than straight bar and larger ductility, although lower stiffness was observed due to effects of splitting cracks.

3.6 REFERENCES

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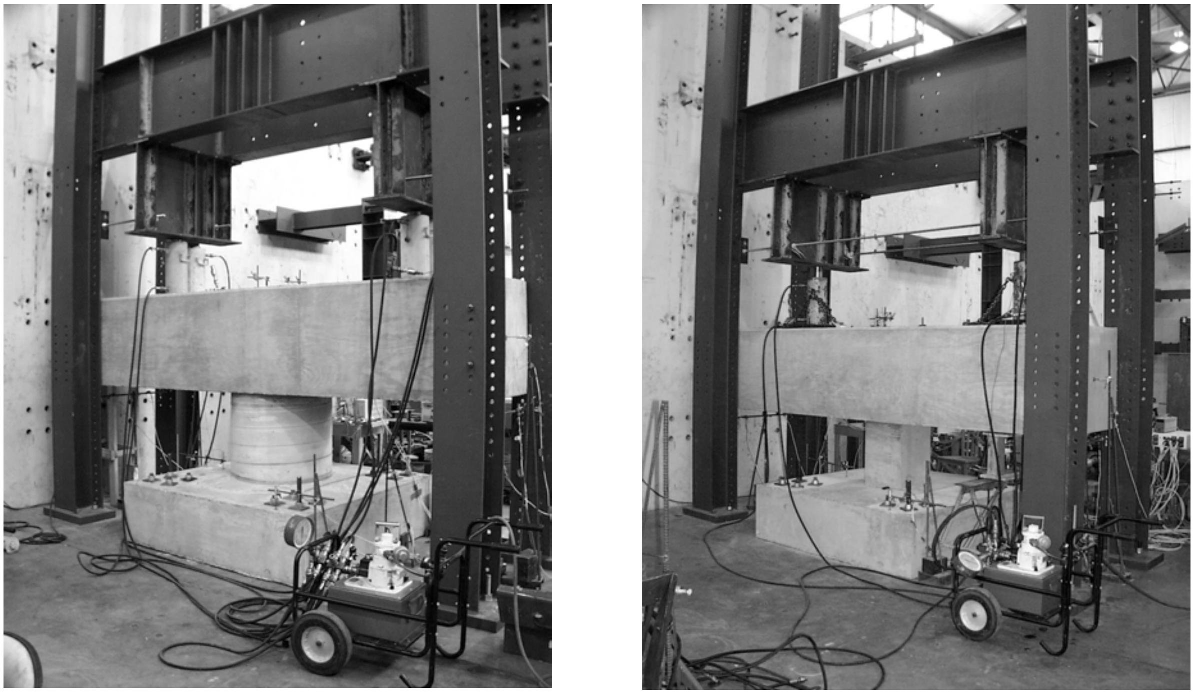
CHAPTER 4:
PHASE 2 CONNECTION TESTS

4.1 INTRODUCTION

Based on Phase 1 test results, Phase 2 investigated precast bent cap-to-column connection behavior through a full-scale, beam-column test for each of the following connection types: 1) single-line grout pocket, 2) double-line grout pocket, 3) grouted vertical duct, and 4) bolted connection. The objectives of Phase 2 were to assess constructability, investigate the influence of the bedding layer on grouting and behavior, and determine the behavior of candidate connection details under service and factored proof loads, as well as failure loads. Figure 4.1 shows representative specimens and the test setup used in the four Phase 2 tests. Phase 2 included fabrication of a precast cap and column stub, cap placement, connection grouting, and loading. This process enabled various constructability and behavioral issues not addressed in Phase 1 tests to be investigated, including:

- construction tolerances
- shim placement and stability during cap setting
- grouting of the bedding layer, precast voids, and multiple pockets and ducts
- connection behavior under realistic service-level and factored loads
- connection failure modes
- potential load concentration at shims
- bedding layer response



a. Trestle Pile Bent *b. Cast-in-Place Column Bent*

Figure 4.1 Phase 2 Test Specimens

By addressing these issues, Phase 2 tests served to verify the adequacy of representative connection details and provide a further basis for development of a design methodology.

This chapter summarizes the Phase 2 connection test program. First, a synopsis of the test program is provided, including a discussion of the test matrix, specimen fabrication, cap placement, grouting, instrumentation, test setup, and test procedure. Then, test results are discussed for each connection type. An additional discussion of Phase 2 tests can be found in Reference 4.1.

4.2 SYNOPSIS OF TEST PROGRAM

4.2.1 Test Matrix

4.2.1.1 Background

Table 4.1 shows the Phase 2 test matrix for the four connection tests. Phase 1 test results were instrumental in helping to define the Phase 2 test matrix. For example, Phase 1 tests indicated that both grout pocket and grouted vertical duct connections will likely possess suitable behavioral characteristics for a precast bent cap system. Therefore, single-line and double-line grout pocket and grouted vertical duct connections were selected for testing. Bolted connections were not previously tested in Phase 1, but were also investigated. In addition, decisions regarding connector type, confining reinforcement, embedment depth, and grout type were based on Phase 1 results. In many instances, minimum provisions suggested by Phase 1 results were used to proportion Phase 2 specimens to test the adequacy of those provisions for actual connection behavior. The following sections explain the use of the parameters summarized in the test matrix.

4.2.1.2 Bent Type and Dimensions

Phase 2 tests investigated connection response for two bent types, a trestle pile bent and bents using a precast cap on cast-in-place columns. As indicated in Section 2.2, these are the primary bent types for application of a precast bent cap system. To maximize comparison between tests and minimize construction time, a single bent cap size was selected for all tests: 2 ft.-6 in. x 2 ft.-9 in. x 12 ft. The size of the cross section was based on a cast-in-place bent designed by TxDOT [4.2]. The 30 in. x 33 in. cross section is considered to be a reasonable compromise for testing, as it is a little larger than most caps used in trestle pile bents and a little smaller than most caps on cast-in-place columns. The cap length of 12 ft. allowed placement of rams at least one beam depth away from the face of the column or pile, to represent load effects for girders distributed over the length of the cap. This was designed to produce load transfer into the column through flexure and shear and to minimize load transfer directly into the column. In addition, the cap length and width accommodated the application of loads at maximum transverse and longitudinal eccentricities for rectangular and inverted tee caps. The overall cap dimensions also enabled Phase 1 formwork to be reused.

Pile and column sizes were also based on typical practice. A 16-in. square pile and 30-in. round column are smaller, but still common, sizes used in bents. The column size was also limited by the large factored load (280 kips on each side of the column) and the capacity of the bolt groups in the test floor. The use of 2-ft. high pile and columns maximized load transfer through the connection and also limited the height of the specimen to facilitate testing.

4.2.1.3 Connection Type and Configuration

Four connection types were tested. The single-line grout pocket on a trestle pile bent is designated as PSL. The other three connection types—double-line grout pocket, grouted vertical duct, and bolted connection—were used with a cast-in-place column, and are therefore designated CDL, CVD, and CBC, respectively.

Table 4.1 Phase 2 Test Matrix

Test		Connection			embed	Grout		Shims	Loading		
ID	Bent	type	connectors	confinement	(in)	brand	method		Vert.	Horiz.	Eccentricity (in)
PSL	Trestle Pile	single line grout pocket	1/3 straight #9's @ 15 in. embedment	#3 @ 4 in. 18-in. diameter	15	EHF	bucket	two 3 x 3 in. steel shims	Serv: 70K Ult: 100K	Serv: 20K Ult: 20K	Transverse: 3.75, 6.75; Longitudinal: 3, 6, 9
CDL	CIP Column	double line grout pocket	2/2 straight #9's @ 15 in. embedment	#3 @ 4 in. 24-in. diameter	15	Sika 212	bucket	two 3 x 3 in. steel shims	Serv: 200K Ult: 285K	Serv: 40K Ult: 40K	Transverse: 4.25, 7.25 (55.5, failure); Longitudinal: 6
CVD	CIP Column	vertical duct	2/2 straight #9's @ 15 in. embedment	#3 @ 4 in. 24-in. diameter	15	MF928	tremie tube	two 2 x 4 in. plastic shims	Serv: 200K Ult: 285K	Serv: 40K Ult: 40K	Transverse: 4.25, 7.25 (55.5, failure); Longitudinal: 6
CBC	CIP Column	bolted	2/2 1-in. diam. bolts @ 15 in. embedment	#3 @ 4 in. 24-in. diameter	15	MF928	tremie tube	two 2 x 4 in. plastic shims	Serv: 200K Ult: 285K	Serv: 40K Ult: 40K	Transverse: 4.25, 7.25 (55.5, failure); Longitudinal: 6

EHF=Euclid Hi-Flow, neat

Sika 212=Sika 212, neat

MF928=Masterflow 928, neat

Because trestle pile bents typically develop a small force transfer through connections, a single-line grout pocket was used in PSL. To investigate potential differences in constructability and force transfer for piles embedded into the cap (versus surface flush with the cap bottom), the single-line grout pocket specimen had a 3.5-in. deep recess at the underside of the cap (Figure 4.12b). Connectors were configured in a 1/3 arrangement (single row of three connectors), as preliminary calculations suggested a minimal number of connectors would be required to transfer factored level connection forces. Figure 4.12a shows the three connectors at 3.5 in. on center at the pile top.

Because Phase 1 grout pocket tests indicated that excellent interlock developed between the grout and concrete, smaller tapers were used in Phase 2 grout pockets. This resulted in minimal impact on placement of longitudinal reinforcement in the cap and limited the grout volume. Table 4.2 summarizes the taper angle and available tolerances for Phase 2 connections. The transverse direction always refers to the direction in the plane of the bent, whereas the longitudinal direction is the out-of-plane direction (i.e., along the length of the superstructure). As shown in the table, a smaller approximately 4-degree taper was used through the cross section of the cap for the single-line grout pocket specimen, compared to the 5-degree taper used in the Phase 1 specimen. Tolerances of +/-2.4 in. in the longitudinal direction and +/-1.9 in. in the transverse direction were provided.

The remaining three connection types were used in CDL, CVD, and CBC tests. Each test used a configuration in which the cap was set on top of shims at the column top and a 2/2 connector arrangement. Although additional connectors can be used in connections, the 2/2 configuration was expected to provide sufficient strength. Double-line grout pockets used a taper of approximately 2 degrees in both directions, in contrast to the 7-degree taper along the beam and 1-degree taper through the cross section in Phase 1. This provided tolerances of +/-0.9 in. and +/-1.4 in. in the longitudinal and transverse directions, respectively. Vertical duct connections used the same 4-in. diameter steel corrugated ducts as in Phase 1. Bolted connections used 3-in. diameter steel corrugated ducts to compare constructability with the CVD specimen. Tolerances were +/-1.4 in. and +/-1 in. for the CVD and CBC ducts, respectively.

4.2.1.4 Connector Size, Anchorage, and Embedment Depth

Based on Phase 1 results, grout pocket and grouted vertical duct tests used straight #9 epoxy-coated bars embedded 15 in. ($13d_b$), or half the cap depth. Number 9 bars represent realistic connector sizes. Straight bars, rather than headed bars, were used to investigate anchorage response. The embedment depth was based on Phase 1 pullout tests, which demonstrated that epoxy-coated, straight #6, #8, and #11 bars achieved yield within $13d_b$ (based on an average concrete compressive strength of approximately 5.4 ksi) due to confinement effects of the concrete surrounding grout pockets and ducts. Although an embedment depth of $13d_b$ was used, a much larger depth was available for embedment. However, the depth was limited to 15 in. to verify anchorage capacity. Preliminary calculations indicated that eccentricities associated with proof tests would not be sufficient to cause yield of connectors in tension. However, failure tests were expected to assess anchorage at yield.

For comparison with CDL and CVD, bolted connections employed 1-in. diameter threaded rods with a specified yield strength of 75 ksi (90 ksi actual). Anchorage was provided by both bond along the rod and bearing at the cap top.

4.2.1.5 Confining Reinforcement

Phase 1 single-line grout pocket tests demonstrated that, at an embedment depth of $8d_b$, confining reinforcement increased strength and ductility and limited service-level cracking. Due to low cost and ease of construction, spiral reinforcement was used in Phase 2 to confine both grout pockets and ducts. Spiral reinforcement was selected over welded wire fabric because spirals in Phase 1 tests exhibited effectiveness at smaller bar displacements. In addition, spirally-reinforced specimens exhibited smaller crack widths and a stiffer response. However, instead of the tight 2.5-in. spacing of #3 spirals used in Phase 1, Phase 2 used #3 spirals at an increased spacing of 4 in. The height of spiral reinforcement was limited to the embedment depth for grout pocket connections, but was extended over the entire height of ducts for CVD and CBC specimens.

4.2.1.6 Grout

The three grout brands tested in Phase 1—Masterflow 928 (MF928), Euclid High Flow (EHF), and Sika 212—were again used in Phase 2 to investigate additional concerns such as different grouting approaches, grouting of the bedding layer, and grouting of multiple pockets or ducts. Based on comparable performance between neat and extended grouts in Phase 1 testing, Phase 2 used only neat grouts. In Phase 1 tests, MF928, EHF, and Sika 212 grouts exhibited significant variations in strength and grouting performance. Although EHF demonstrated lower strength and working time, as well as potential durability concerns related to an unusually porous surface and more extensive cracking, EHF was used in PSL to investigate the ability of a grout to transfer load even if other characteristics are questionable. Sika 212 was used in CDL to investigate performance in a grout pocket connection, as Sika 212 was only used for a grouted vertical duct test in Phase 1 (VD05). MF928 was selected for grouting multiple ducts in CVD and CBC. EHF and MF928 grouts were supplied by the manufacturers, and Sika 212 was purchased locally.

4.2.1.7 Grouting Methods

Two different gravity-flow grouting methods were used: a bucket approach for grout pockets (PSL and CDL), and tremie-tube approach for ducts (CVD and CBC). Although Phase 1 did not address grouting of the bedding layer or multiple pockets or ducts, these two approaches were considered to be viable approaches for a precast bent cap system. The bucket method simulated a simple field grouting approach using 5-gallon buckets to fill grout pockets. CVD and CBC ducts were grouted using a funnel and hose, with the hose extended to the bottom of one of the ducts during grouting of all ducts. In contrast to the modified tremie-tube grouting in Phase 1, continuous grouting was conducted to minimize air entrapment. Grout pumping is another viable option and is discussed in Chapter 7.

4.2.1.8 Shims

Shim material and placement were investigated in Phase 2. It is anticipated that precast caps will be supported by shims or friction collars in the field. Although steel and plastic shims are commonly used in practice, steel shims pose concerns related to potential hard points at the cap-to-column interface, and corrosion. To investigate potential differences in response, Phase 2 tests used both types of shims. Shim areas were limited to 7% of the pile cross section and 2.5% of the column cross section. Placement of shims was also addressed, with regard to grouting of the 1.5-in. thick bedding layer.

4.2.1.9 Loading

As shown in Figure 4.1, rams were positioned on both sides of a connection to impose realistic connection forces corresponding to dead, live, and wind loads at service and factored levels. By using multiple rams and changing the ram eccentricity, connection forces were varied to produce connection forces associated with transverse and combined transverse and longitudinal eccentricities (Figure 4.2). The east ram was placed a distance equal to an effective cap depth away from the edge of the column (i.e., approximately 41 in. from the center of the connection). For simplicity, this distance was maintained for all specimens, despite the smaller pile size in PSL. The west ram was placed a distance, x_2, away from the center of the connection, where x_2 was determined by the following:

$$x_2 = (2 \times e_{transv}) + x_1$$

where:

e_{transv} = eccentricity in transverse direction

x_1 = distance of east ram from center of connection

The factor of two accounts for the fact that half of the total vertical load at the column top was provided on each side of the connection, as shown below:

$$e = \frac{M_{conn}}{P_{total}} \; ; \; M_{conn} = P_{ram} \times (x_2 - x_1)$$

where:

M_{conn} = connection moment, i.e., moment at the column top

P_{total} = total ram load at column top

P_{ram} = total ram load on one side of a connection

Table 4.1 shows the range of vertical and horizontal loads and eccentricities used in testing. Loads and eccentricities were based on TxDOT experience and a series of analyses of typical trestle pile and cast-in-place column bents using rectangular and inverted-tee caps [4.3,4.4]. Experience and analyses indicated that the maximum realistic eccentricity at the column top in either direction was approximately 6 in. for column bents and smaller for pile bents. For combined eccentricities, a value less than 6 in. would be expected in one of the two directions. Service level vertical loads corresponded to 70% of factored levels. The load series for each connection was selected to progressively test the connection to more severe load combinations. After factored (proof) loads were applied, a large failure eccentricity was applied, usually by loading a single ram on only one side of the connection (Figure 4.2). This produced yield in one or more connectors, allowing connection anchorage, strength, ductility, and stiffness to be further investigated. Section 4.2.8 discusses details of the load sequence used in testing.

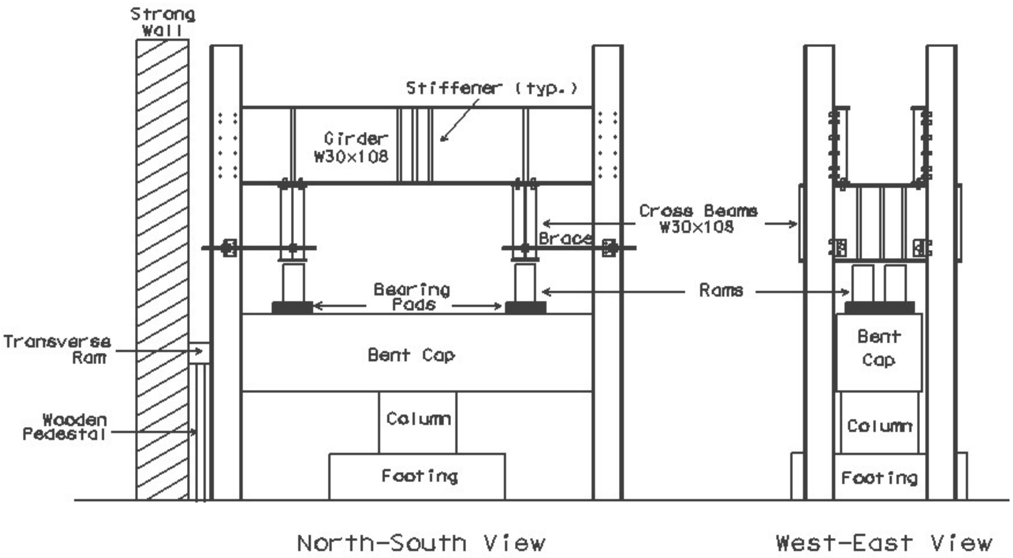


Figure 4.2 Phase 2 Test Setup [4.1]

4.2.2 Specimen Fabrication

Connection tests were conducted using specimens that consisted of two elements: a bent cap (beam) and a column or pile stub cast monolithically with a footing.

4.2.2.1 Trestle Pile Bent Specimen

Figures 4.3 and 4.4 show plan, elevation, and section views of the cap, footing, and pile for the trestle pile bent specimen. As mentioned previously, the bent cap was a 2 ft.-6 in. x 2 ft.-9 in. x 12 ft. beam, representing a full-scale trestle pile cap. Longitudinal reinforcement ratios were based on a recent TxDOT design [4.2]. Cap reinforcement was checked to ensure flexural or shear failure did not occur before factored loads were applied to the connection.

Figure 4.5 shows the reinforcement cage used for the cap. Phase 1 beam forms were reused for Phase 2 caps. In contrast to the Phase 1 wooden insert, a Styrofoam insert was used to form the more complex grout pocket shape (Figure 4.5b). Three Styrofoam pieces, one for the base and two halves for the pocket, were cut with a hot wire and glued together to form the required shape. Two voids were also cut into the base, which allowed the insert to be placed over shear tabs connected to the floor formwork. This prevented movement of the base of the insert during casting. Figure 4.5b also shows the confining reinforcement extending to a height equal to the embedment depth (15 in.).

Figure 4.6a shows the 5 ft.-8 in. x 5 ft.-8 in. x 18 in. footing formwork and cages for the footing and pile. Four templates were attached to the base of the footing formwork to accurately align PVC ducts for connecting the footing to the test floor. Instrumented connectors in the 1/3 configuration are shown extending above the pile reinforcement. A wooden template was used at the top of the connectors to hold the bars in place (Figure 4.6b). Instead of using high strength strands, pile reinforcement consisted of 8-#8 bars with #3 ties at 3 in.

Figure 4.6a shows the footing formwork in position prior to casting of the footing. Because the pile was cast with the same batch of concrete as the footing, additional formwork was placed on top of the footing to stabilize the wet footing concrete and support the pile formwork after the footing was cast (Figure 4.6b).

4.2.2.2 Cast-in-place Column Bent Specimens

The CDL, CVD, and CBC specimens used the same reinforcement for the bent cap, footing, and column stub. The only difference between specimens was the Styrofoam inserts used in the CDL cap and the corrugated ducts used in CVD and CBC caps. Therefore, discussion of the fabrication for these specimens is limited to the CVD specimen.

Figures 4.7 and 4.8 show the plan, elevation, and section views of the cap, footing, and pile for the CVD specimen. Connector configurations for the CDL and CBC cases are also shown in Figure 4.7. Because the same cap dimensions were used for cast-in-place bents as for the trestle pile bent, significantly larger amounts of longitudinal and shear reinforcement were required to resist the vertical loads that were three times larger than PSL loads.

Figure 4.9 shows the reinforcement cage used for the cap. Although such a cage was not realistic for a bent cap, two layers of #11 longitudinal bars and double #6 stirrups were required to ensure flexural and shear failures were precluded during testing. However, the use of a single bottom layer of longitudinal reinforcement and an embedment depth of only 15 in. ensured realistic conditions in the connection region for CDL and CVD specimens. Minimal impact is expected for the CBC specimen as well, since transfer of forces occurs both along the bolt as well as at the top anchorage. Figure 4.9b shows the confining reinforcement extending the entire depth between longitudinal reinforcement. The CBC specimen also used confinement throughout the entire height, while the CDL specimen used confinement over a height equal to the embedment depth (15 in.). Tolerances for duct placement were typically +/-3/8 in. Ducts were sealed at both ends to prevent concrete from entering the ducts during casting. Figure 4.10 shows the formwork and cages used for the footing and column. The 30-in diameter column form is shown in Figure 4.10b, with connectors in the 2/2 configuration.

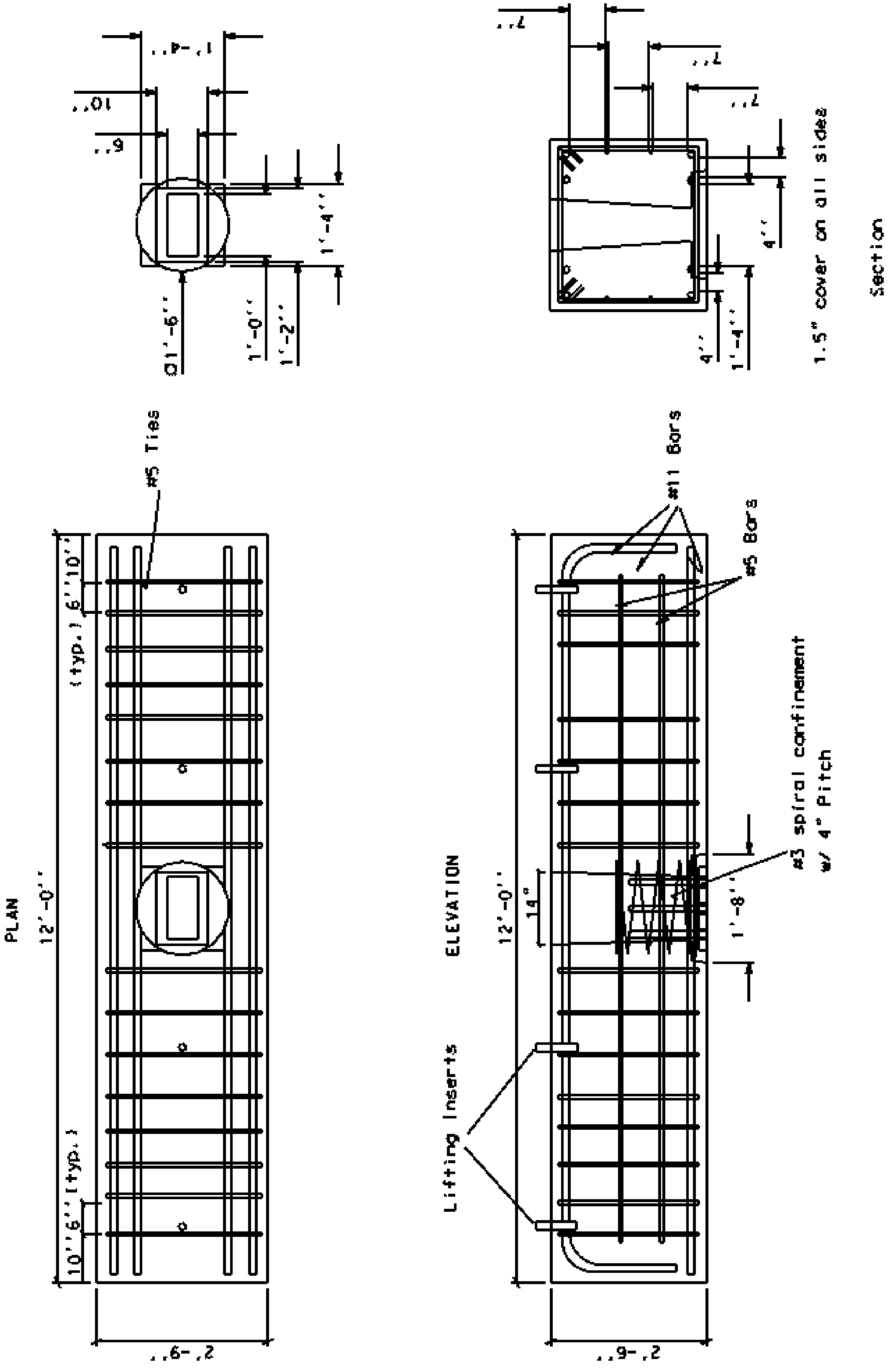


Figure 4.3 Trestle Pile Bent—Precast Cap Schematic

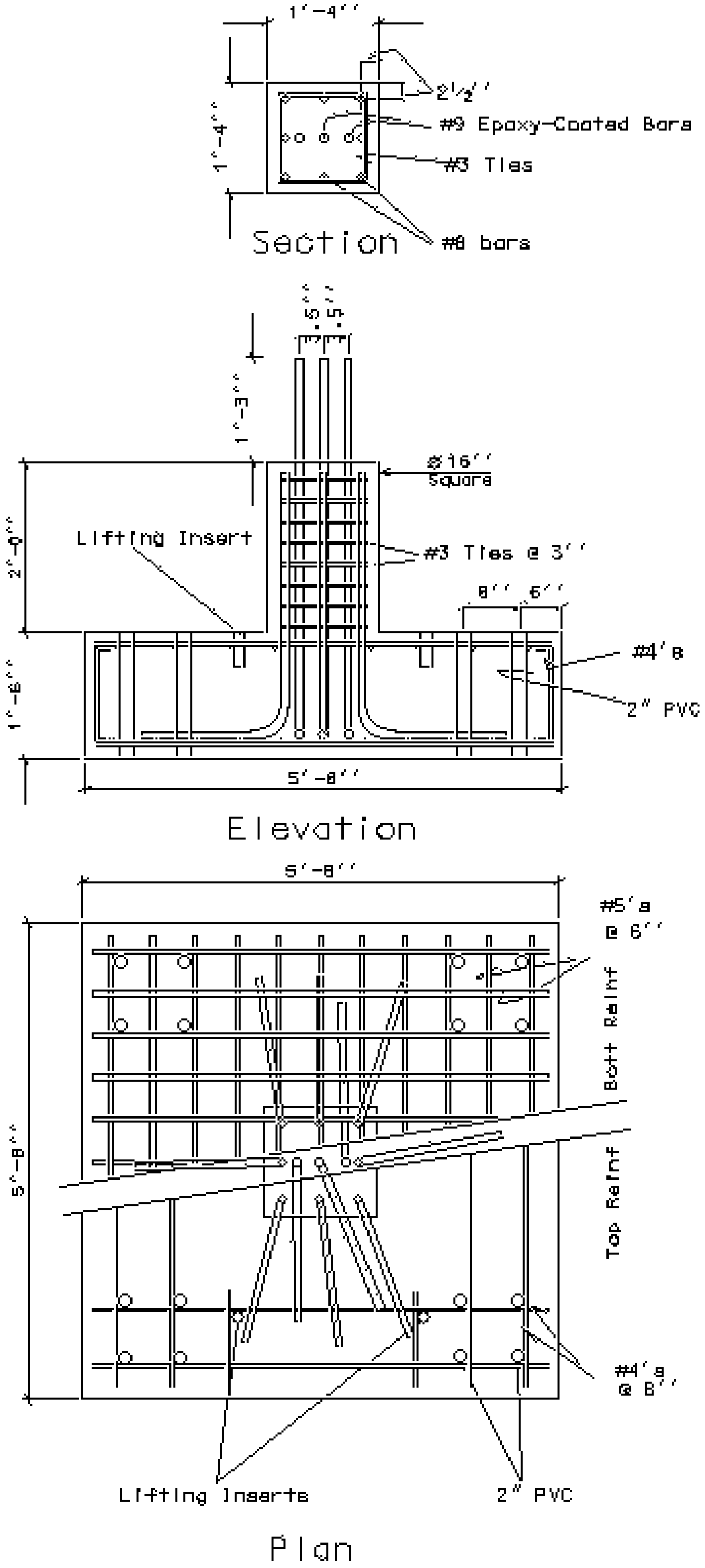
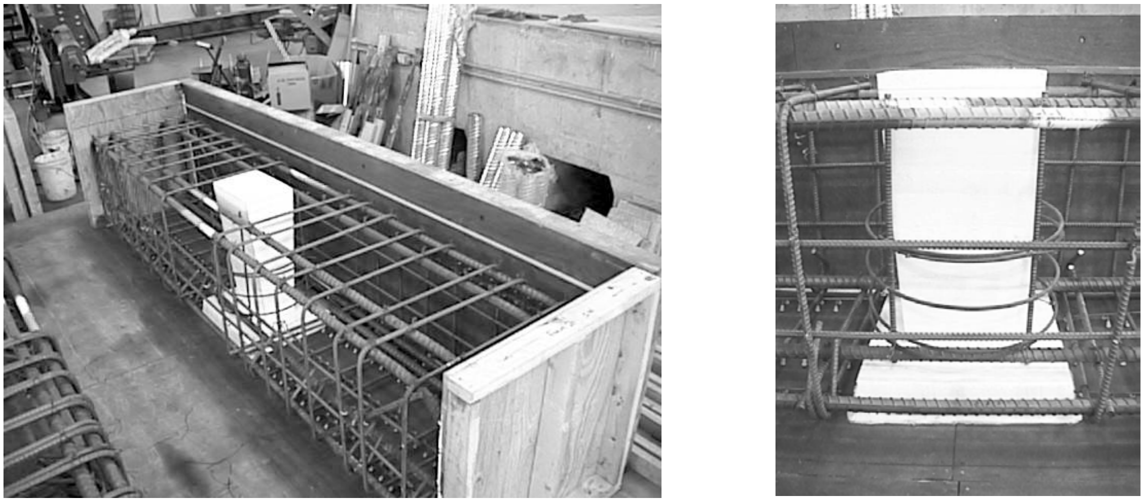


Figure 4.4 Trestle Pile Bent—Footing and Pile Stub Schematic



a. Rebar Cage *b. Styrofoam Insert*

Figure 4.5 Trestle Pile Bent—Precast Cap

a. Footing and Pile Cage *b. Formwork Prior to Casting*

Figure 4.6 Trestle Pile Bent—Footing and Pile Stub

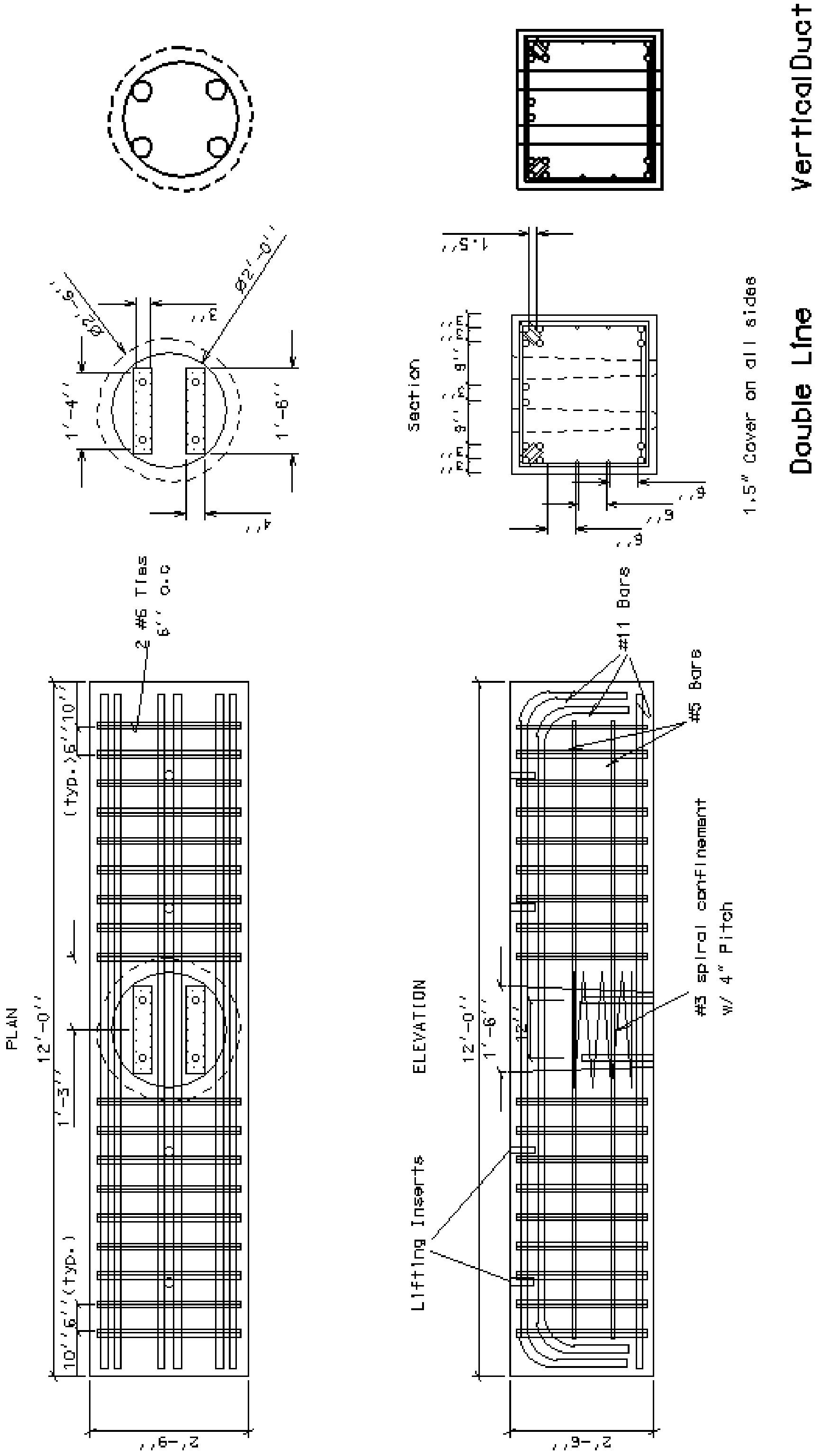


Figure 4.7 Cast-in-place Bent—Precast Cap Schematic

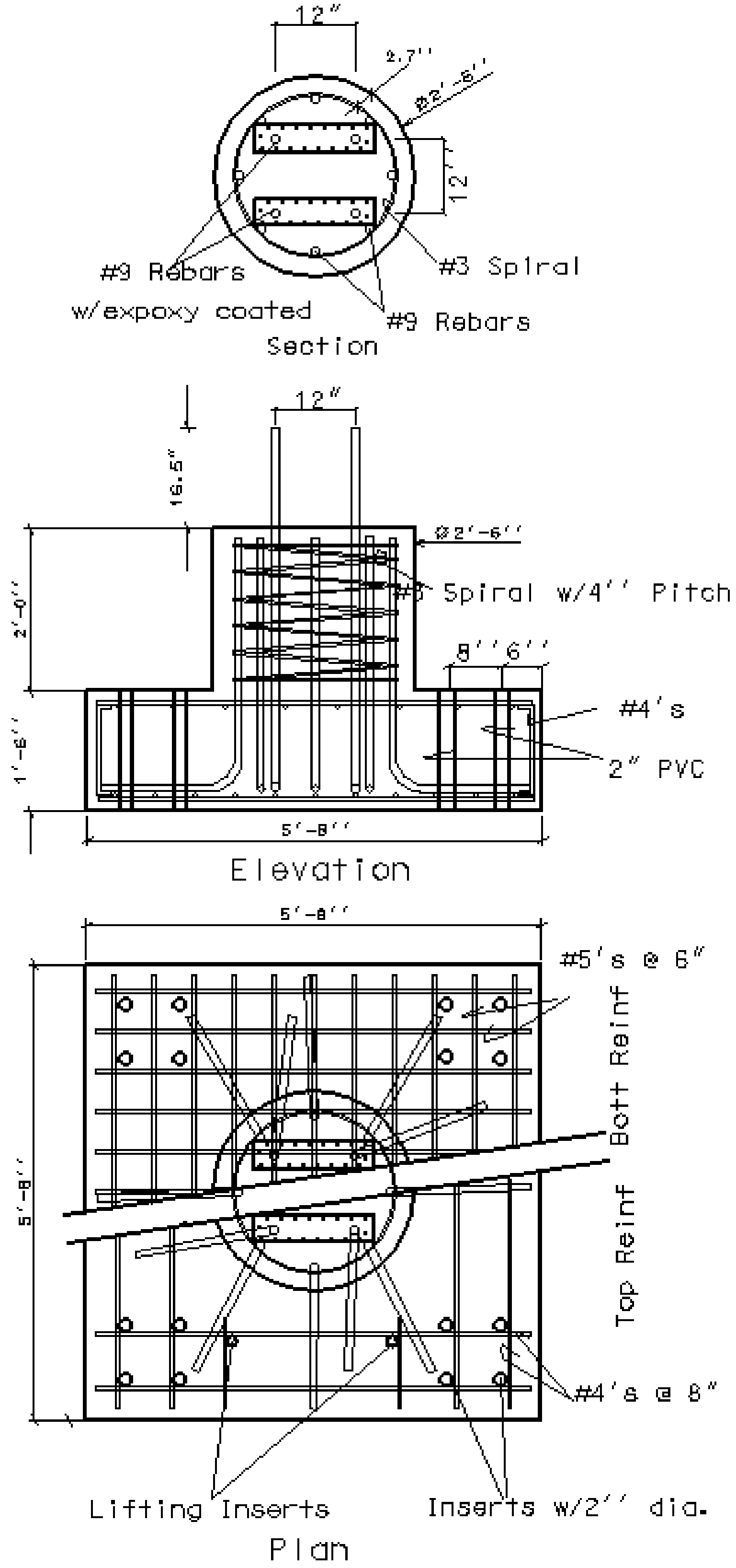


Figure 4.8 Cast-in-place Bent—Footing and Column Stub Schematic

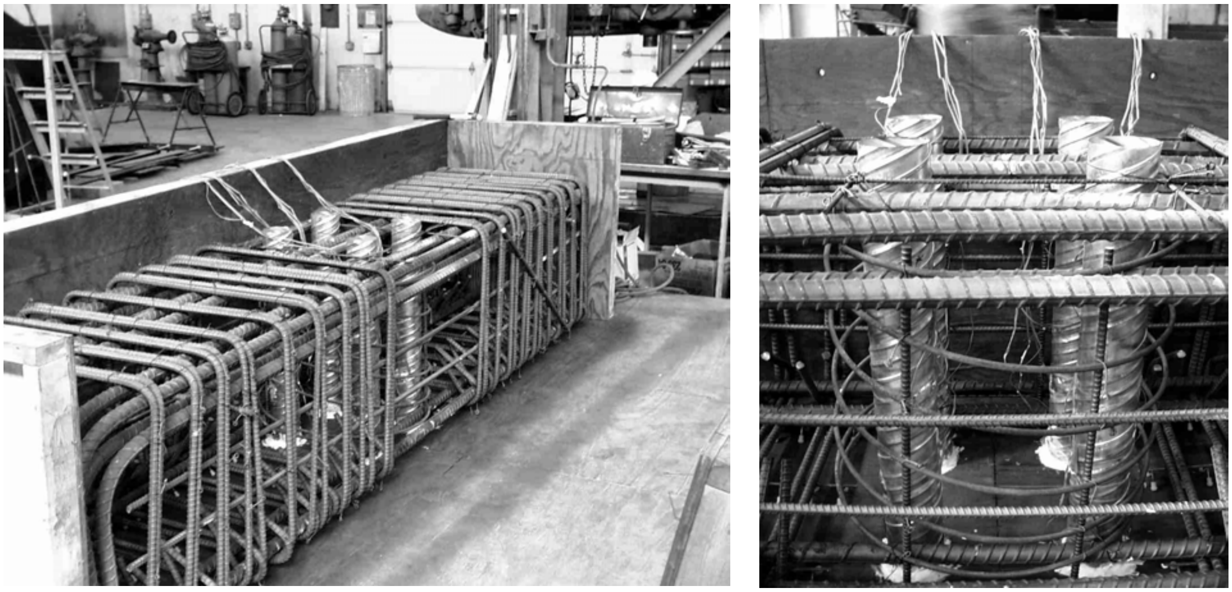


Figure 4.9 Cast-in-place Column Bent—Precast Cap (Vertical Duct)

a. Rebar Cage

b. Column Form

Figure 4.10 Cast-in-place Column Bent—Footing and Column Stub

4.2.2.3 Casting

Differences between Phase 1 and Phase 2 casting were primarily related to the footing. Figure 4.11a shows the footing surface being finished. Following this step, formwork was secured to the top of the footing for casting the column concrete (Figure 4.11b). Concrete was shoveled in column forms (Figure 4.11c), vibrated at several intervals, and finished. In some cases, a slight uplift between the three footing top forms occurred during vibration of the column concrete. In one case (CVD), this caused the column to tilt slightly, although the connectors remained plumb and the top surface of the column was level. Test results did not appear to be influenced by this. For the PSL and CDL specimens, Styrofoam inserts were removed using a knife and gasoline. After form removal, pockets and some ducts were air blown or scrubbed to remove debris.

a. Casting Footing *b. Attaching Top Form* *c. Column Form*

Figure 4.11 Phase 2 Casting of Footing and Column Stub

4.2.3 Cap Placement

Following casting, footing specimens were bolted to the test floor and shims were placed in preparation for cap placement. Cap placement operations are outlined for the PSL and CBC specimens.

4.2.3.1 Trestle Pile Bent Specimen

Figure 4.12a shows the footing and 16-in. square pile with a single-line arrangement of three epoxy coated #9 connectors (1/3). Two steel shim stacks are shown in place at the top of the pile in the longitudinal direction. Shims were 3 in. x 3 in. and 1.5 in. thick and consisted of three ½ in. steel plates tack-welded together. Shims were placed 1-1/2 to 2 in. away from the edge of the pile and required slight adjustment to ensure a level bearing for the cap. Shims were not held down and did not move during cap placement.

Figure 4.12b shows the cap specimen lifted with a crane into position for placement. The block-out is shown from the bottom of the cap, including a recess and sides that taper in both directions. Although the connectors were placed slightly off-center during casting of the pile, they still easily fit within the grout pocket (Figure 4.14a). The 15-in. embedment depth was measured as the distance into the pocket void beyond the recess. Air vent tubes were used during grouting to eliminate air that could be trapped on the horizontal "shelf" on which the shims rest.

Figure 4.12c shows the cap in final position, resting on the shims. Three posts were placed under the cap to provide additional stability of the cap during grouting. Two posts were jacked up slightly to help level the cap. The footing is shown tied to the strong floor using four bolt groups.

4.2.3.2 Cast-in-place Column Bent Specimens

Figure 4.13a shows the footing and 30-in. diameter column with a 2/2 arrangement of 1-in. threaded rods. Two plastic shim stacks were used in setting the cap. Shims were 2 in. x 4 in. x 1-1/2 in. and consisted of several layers of plastic glued together. Shims were placed approximately 5 in. away from the edge of the column and were glued to the surface to prevent movement during cap placement. Figure 4.13 also shows a small ring of the column form used to form the 1.5-in. thick bedding layer, as well as two 3/8-in. inner diameter vent tubes used to vent air during grouting. In contrast to the PSL specimen, CDL, CVD, and CBC caps were set above the column and therefore left the bedding layer exposed.

a. Pile Stub with Shims *b. Cap Setting* *c. Cap Resting on Shims*

Figure 4.12 Cap Placement Operation for Trestle Pile Bent

a. Column Stub *b. Cap Setting* *c. Uneven Underside of Cap*

Figure 4.13 Cap Placement Operation for Cast-in-place Column Bent

Figure 4.13b shows the cap being lowered over the bolt group. Unlike Pierce Street Elevated construction, pipes were not used to help thread the cap over connectors. However, such an aid is recommended for field operations. The top of Figure 4.13c shows an uneven surface at the underside of the bent cap (due to warped beam forms), which caused the cap to tilt when placed on shims. Grinding of the bottom surface and adjustment of the posts eliminated tilting. Warping on the underside of the CVD cap also required grinding.

Figures 4.14a, 4.17b, and 4.20a show the final position of the connectors for PSL, CDL and CBC, respectively. Connectors were placed within available tolerances. As shown in Table 4.2, CDL and CBC connections provided the smallest tolerances (0.9 in. and 1.0 in., respectively). Based on fabrication and cap setting operations, minimum tolerances should be approximately 1 inch, although a 1.5-in. tolerance is preferable.

a. Plan View of Pocket *b. Bottom of Pocket* *c. Pouring from Bucket*

Figure 4.14 Grouting Operations for Single Line Grout Pocket—1

Table 4.2 Connection Taper and Tolerances

Test	**Connection Type**	**Taper[A](degrees)**		**Tolerances[A] (in.)**	
		Longitudinal	**Transverse**	**Longitudinal**	**Transverse**
PSL	Single line grout pocket	3.8	1.9	2.4	1.9
CDL	Double line grout pocket	1.9	1.9	0.9	1.4
CVD	Vertical duct	0	0	1.4	1.4
CBC	Bolted connection	0	0	1.0	1.0

A. Transverse direction refers to the in-plane direction of the bent. Longitudinal direction is perpendicular to the transverse direction.

4.2.4 Grouting Operations

Similar to Section 3.2.3.1, grouting operations for all connections included forming of the bedding layer, prewatering and vacuuming, gravity-flow grouting, and curing. However, there were substantial differences for each connection type, due to the use of different grout types, connection configurations, and grouting methods. The following sections describe important details and results of grouting. Grouting operations for the CVD and CBC specimens are discussed together because the same grout and grouting operations were used. Lessons learned are summarized at the end of the section.

4.2.4.1 Single-line Grout Pocket

Euclid Hi-Flow (EHF) neat grout was used for the PSL specimen. EHF was considered to be potentially more problematic than other grouts, as Phase 1 tests with EHF grout exhibited lower strength and working time, as well as potential durability concerns related to an unusually porous surface and more extensive cracking.

Figure 4.14a shows the single-line grout pocket prior to grouting, including a taper in each direction. Lead wires were positioned away from the bar to prevent affecting bond. Figure 4.14b shows the bottom of the grout pocket sealed with plywood and caulk. Six ¼-in. diameter vent tubes were used to remove entrapped air at the horizontal shelf. After caulking, the pocket surfaces were pre-soaked to limit absorption of grout water into the cap and to verify sealing of the pocket. Excess water was completely drained from the pocket prior to grouting.

Figure 4.14c illustrates grouting of the pocket with a 5-gallon bucket. To limit air entrapment during grouting, grout was poured against the side wall. Six 50-lb. bags were mixed in a paddle-type mortar mixer with 11 lbs. of water per bag, which was close to the recommended amount to achieve a fluid grout. Grout was mixed using tap water and at an ambient temperature of 75 degrees F. During mixing, an additional ½ lb. of water per bag was added because the mix appeared too stiff. Using the flow cone test, a flow of 57 seconds was measured. Although the grout had a medium-thick "milkshake" consistency, it was considered sufficiently flowable to grout the large pocket opening and bedding layer, and thereby avoid remixing. Figure 4.15a shows the unusually large number of "clumps" that were removed from the mix using a ½-in. mesh when pouring grout from the mixer into 5-gallon buckets. If not removed, such clumps could hinder grouting of the pocket and affect connection strength and durability.

Grout was placed in the pocket in three lifts. The pocket was tamped after each lift using a reinforcing bar (Figure 4.15b). Most vent tubes exhibited a steady flow of grout and air removal (Figure 4.16a), although some tubes showed little air or grout removal. After the pocket was filled, it was decided to use a pencil vibrator (Figure 4.15c), to see if additional air would be removed. Although additional air surfaced to the pocket top and was removed from the bedding layer through vent tubes, the use of a vibrator appeared to unduly agitate the grout volume and possibly even entrap air in the process. Therefore, the use of a vibrator is not deemed essential and should be used with caution.

a. Clumps *b. Tamping of Pocket* *c. Vibrator*

Figure 4.15 Grouting Operations for Single Line Grout Pocket—2

a. Air Venting *b. Air Bubbles at Top* *c. Surface after Grouting*

Figure 4.16 Grouting Operations for Single Line Grout Pocket—3

After grouting, curing compound was sprayed on the pocket top. As was observed in Phase 1 grouting, air bubbles matching bar locations appeared at the surface soon after grouting (Figure 4.16b). One day after grouting, a #2 bar was pushed down through the void above the center bar and penetrated 2 in.

(Figure 4.16c). The top ½ in. of the surface had a soft porous texture. The exposed bottom surface of the grout indicated that the grout filled the void around the connection completely.

For inspection purposes, a 3-in. diameter core was taken from the PSL, CDL, and CVD specimens after testing (Figure 4.23). Cores were approximately 5 in. in length and were taken directly above the bar subjected to the greatest tension during testing. The EHF core shown in Figure 4.23a revealed a clear discontinuity between the porous top layer and the dense grout beneath. Red dye was found to penetrate only this top layer.

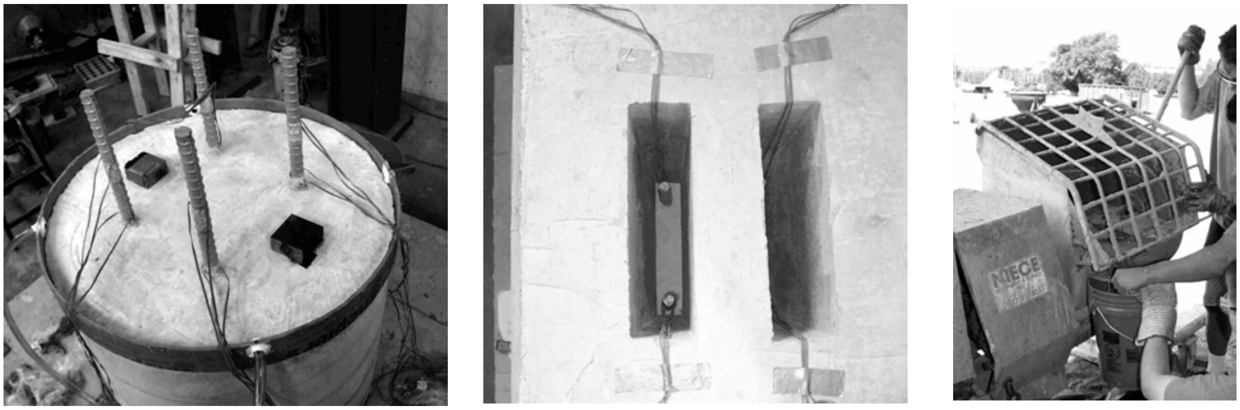
One-day grout cube strengths were only 1.4 ksi. Table 4.3 summarizes key parameters related to the different Phase 2 grouts and reveals a low relative strength and larger amount of entrapped air for EHF compared to other grouts.

Table 4.3 Phase 2 Grouting Data

Test	Grout Brand	Water (lbs/bag)	Temp (F)	Flow (sec)	Air Voids at Surface	Grout Cube Strength (ksi)	Notes
PSL	EHF	11.5	75	57	Several large voids	4.0 (22-day)	Porous surface, large clumps
CDL	Sika 212	10	81	17	None	5.1 (6-day)	Segregation, gap at bedding
CVD	MF928	10	81	34	None	5.2 (6-day)	
CBC	MF928	10	95	53	Small bubbles adjacent to bolts	6.1 (6-day)	

4.2.4.2 Double-line Grout Pocket

Sika 212 was used to grout the double-line pocket. Figures 4.17a and 4.17b show the configuration of the double-line grout pocket prior to grouting, including the use of four vent tubes with a ½-in. inner diameter. Table 4.3 provides pertinent grouting information.



a. Bedding Layer & Shims *b. Plan View of Pocket* *c. Grout Mixing*

Figure 4.17 Grouting Operations for Double Line Grout Pocket—1

During mixing, the grout appeared very fluid. However, considerable segregation between the fine aggregate and paste developed. This resulted in very little aggregate being poured from the bucket into the flow cone, leading to a flow of just 14 seconds. The flow cone test was conducted a second time with a more representative amount of aggregate placed in the cone. This increased the flow to 17 seconds. The grout consistency in the bucket was very watery on top, but sandy and dense at the bottom. Figure 4.17c shows grout being poured into 5-gallon buckets. The bucket was lifted up to the mixer to minimize air entrapment. With such a fluid grout, air was easily removed from the mix.

Figure 4.18 illustrates grouting operations. Because of the narrow pocket widths, grout was poured against a piece of plywood to direct grout into the pocket with minimal air entrapment. Figure 4.18a shows the effect of segregation. Fluid at the top of the bucket poured out, but fine aggregate stuck to the bottom of the bucket and had to be removed by hand (Figure 4.18b). This occurred even though initial grouting took place well within 15 minutes of mixing. At later stages of grouting, the segregated aggregate began to harden at the bottom of the bucket, making it difficult to complete grouting operations. This also required two additional bags of grout to be mixed to provide sufficient volume to fill the pockets. Figure 4.18c shows tamping of the pocket. Tamping helped consolidate the pocket and bedding, yet because of the density of the aggregate at the bottom, a pencil vibrator was also used. After tamping the first lift, the vent tubes were opened up to allow grout to flow out and remove embedded air at the bedding layer. Grout and air flowed out from three of the four vent tubes at the bedding.

a. Segregation *b. Removal of Sand* *c. Tamping of Pocket*

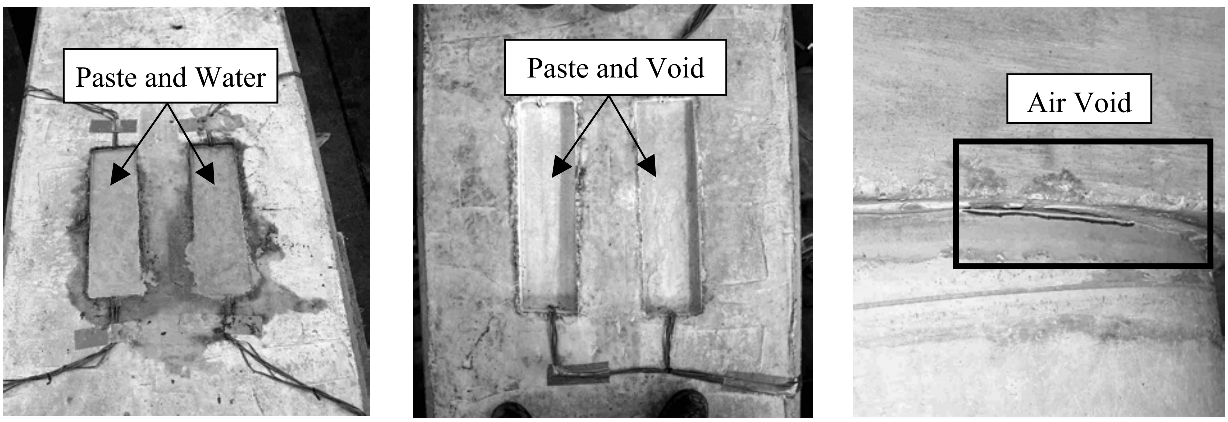
Figure 4.18 Grouting Operations for Double Line Grout Pocket—2

Due to segregation, the level of aggregate in each pocket differed significantly. The level of the sand was 17 in. below the cap top in the north pocket, but only 10 in. below the cap top in the south pocket. Because the top of the connectors was 15 in. below the surface, this did not influence connection behavior.

Two final issues related to segregation produced particular concern for a precast bent cap system: 1) a cavity at the pocket tops, and 2) a void in the bedding layer. As shown in Figure 4.19a, after grouting, water and paste rose to the surface, forming a layer several inches deep. For this reason, curing compound was not used. Within one day after grouting, water in this layer had evaporated, leaving a 2-in. deep cavity at the top of both pockets (Figure 4.19b). If not topped off later with grout and/or sealed, a cavity in an actual system could act as a basin to retain moisture and lead to corrosion of the connection. In addition, the top layers were white and orange and had a very soft, paste-like consistency for a depth of approximately 2 in.

After stripping the bedding form, a ¼-in. by 2.5 in. gap was found at the top of the bedding layer on the south side, as shown in Figure 4.19c. A ruler penetrated into the void over an area of approximately 6 in. by 6 in. Because the loss of bearing was expected to be less than 10% for the various load combinations, the void was not pressure grouted. Based on a similar gap encountered with the use of Sika 212 in Phase

3 grouting, it is believed that segregation led to water pockets at the bedding that left voids after evaporation.



a. After Grouting *b. At 24 Hours* *c. Void at Bedding*

Figure 4.19 Grouting Operations for Double Line Grout Pocket—3

Other potential contributors to formation of the void at the bedding were considered. Air vents at the bedding were initially blocked then opened up. Vents should be kept open at all times during grouting. However, Phase 3 grouting with Sika 212 had vents opened throughout, yet a gap still developed. Another possible contributor to the void may have been the placement of the vent tubes at the mid-height of the bedding layer, rather than the top. This requirement can be easily adopted. The combination of shim placement near the bottom of the pockets and clumps of sand in the grout may have hindered the flow of grout to some extent. However, the gap was not on the side where the vent tube flowed out little grout. Nevertheless, it may be prudent to limit the placement of shims, as well as the size of individual shims and total shim area.

Figure 4.23b shows the Sika 212 core. The top ½-in. layer was a soft paste-like material. Beneath that was a 3-in. layer of grout mixed with paste. Approximately 3.5 in beneath the surface, regular hardened grout appeared. This core broke into a number of pieces due to the difficulty in removing the core.

4.2.4.3 Vertical Duct and Bolted Connection

Masterflow 928 (MF928) neat grout was used for the CVD and CBC specimens. Figure 4.20a shows the CBC ducts prior to grouting, including an offset of the bolts within the pockets. This actually helped provide the needed space for placement of the tremie tube within the duct (Figure 4.20b). This demonstrates the importance of providing adequate tolerances.

Prior to grouting, pockets were sealed and prewatered. Figure 4.20b shows the 4-quart funnel and 1-in. inner diameter tremie tube used for prewatering and grouting. Two vent tubes—one adjacent to the duct used for grouting and the other at the opposite side—were sealed during prewatering and then opened to drain water. In contrast to CDL grouting, these tubes were located at the *top* of the bedding form to better remove air at the bedding layer. However, due to their placement, excess water needed to be vacuumed out of the bedding. An alternative approach is to place secondary water ports at the bottom of the bedding form.

a. Plan View of Ducts *b. Funnel and Tremie Tube* *c. Continuous Flow*

Figure 4.20 Grouting Operations of Bolted Connections—1

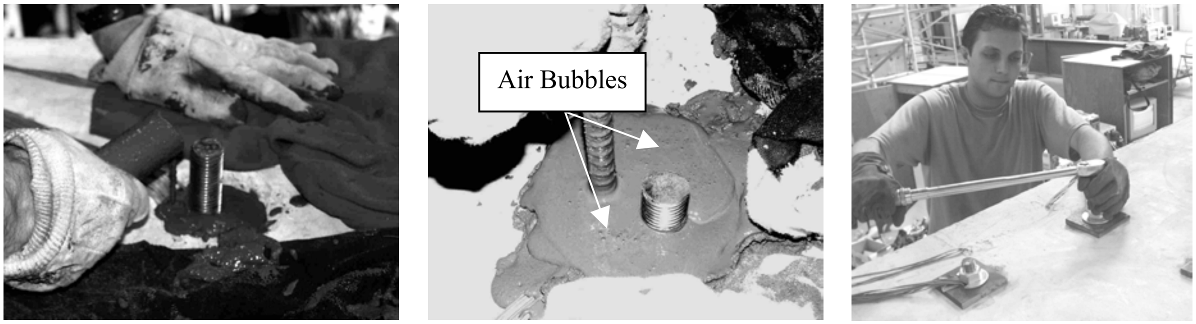
As shown in Table 4.3, mixing of the CVD grout occurred at a temperature of 81 degrees F and produced a fluid grout with a smooth "milkshake-like" consistency and a 34-second flow. CDL grouting occurred at a higher temperature of 95 degrees F, which resulted in a slightly thicker grout consistency and a grout with a longer, but usable, 53-second flow. It should be noted that the grout specification discussed in Chapter 6 requires a flow between 10 and 30 seconds. In practice, cooler water or small additional amounts of water within the manufacturer's recommendations would be required to achieve an adequate flow. Neither was used here.

Figure 4.20c shows tremie-tube grouting for CBC. To fill the connection voids and limit air entrapment within the connection, the tube was lowered to the bottom of one duct and grout was poured from 5-gallon buckets *continuously* until the grout level rose in all the ducts. The two vent tubes were sealed when grout streamed out in a steady flow without air. When the grout level rose within approximately 3 in. of the surface, the tube was removed. The ducts were then topped off from above, as shown in Figure 4.21a. Each duct was tamped with a rod two times during grouting. During grouting of the CVD specimen, air quickly rose to the surface. Hence, little air removal was observed during tamping. However, as shown in Figure 4.21b, the thicker CBC grout exhibited significant air removal both during and after tamping. The 30-minute working time provided a comfortable amount of time for grouting operations.

The top surface of the grout appeared smooth and even for the CVD specimen (Figure 4.22b). However, small air bubbles appeared adjacent to CBC bars. These air voids may have been due to the longer flow time, due in part to the high temperature, which resulted in air bubbles rising more slowly. As seen in Table 4.3, air bubbles appeared at the surface only for grouts with longer flow times (EHF, 57-sec. flow; MF928 (CBC), 53-sec. flow). In addition, the extension of the rods to top of the duct may have facilitated a path for air bubbles to the surface.

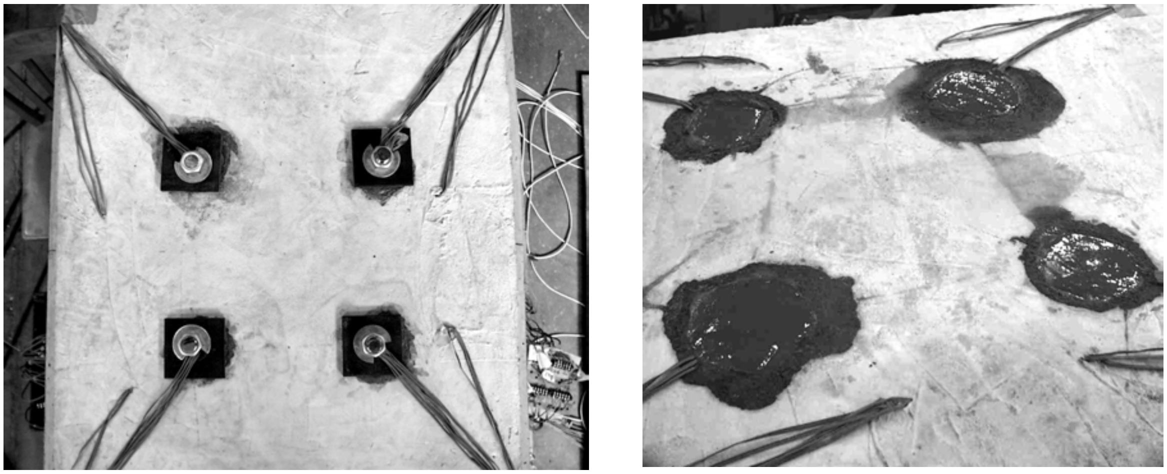
CVD and CBC grout surfaces were cured using curing compound and wet rags. After one day, bolts on the CBC specimen were tightened, as shown in Figure 4.21c. Although bolts were tightened before the grout hardened on the Pierce Street Elevated project, the approach used in CBC permitted monitoring of air bubbles at the grout surface and determining the potential need for a surface sealant. Bolts were tightened to a torque of 200 ft-lbs. Figure 4.22 compares the top surface of the connection for the CVD and CBC specimens.

After stripping the bedding form, bedding layers for both specimens were found to be completely filled without any gaps. Figure 4.23c shows the uniformly dense MF928 grout core taken from the top several inches of a CVD duct.



a. Topping Off Duct *b. Tamping of Duct* *c. Tightening of Bolts*

Figure 4.21 Grouting Operations of Bolted Connections—2



a. Bolted Connection *b. Vertical Duct*

Figure 4.22 Top Surfaces of Ducts

a. Euclid Hi-Flow-PSL *b. Sika 212-CDL* *c. Masterflow 928-CVD*

Figure 4.23 Cores of Top Grout Surfaces

4.2.4.4 Lessons Learned

Several lessons were learned through cap setting and grouting operations. Lessons related to cap setting include the following:

1. Construction tolerances should be at least +/-1 in. in transverse and longitudinal directions, although +/-1.5 in. is recommended. These tolerances should account for combined tolerances associated with placement of connectors in piles or columns and fabrication and placement of pockets and ducts in the bent cap. The tolerances suggested above would require a slightly larger dimension for the double-line grout pocket in the transverse direction, and a minimum duct diameter of 3.25 in. for CVD and CBC specimens. A 4-in. duct diameter is recommended for vertical duct and bolted connection specimens.
2. Both steel and plastic shims are an effective means to support the cap during placement. It is recommended that shims be glued to the surface to prevent shifting. In addition, the underside of the cap should be as smooth as possible to help ensure a proper bearing surface for shims.
3. Pipes should be used to facilitate cap setting over multiple connectors for CVD and CBC connections.

Phase 2 grouting operations reinforced the lessons learned during Phase 1 grouting. The following additional lessons should also be taken into account:

1. Gravity-flow grouting can be used to grout all connection types. The bucket method can be used for grout pockets, and tremie-tube grouting can be used for any connection type. However, to prevent entrapment of air at the bedding layer, vertical duct and bolted connections should use tremie-tube grouting when gravity-flow grouting is used. Precautions should be taken to minimize air entrapment during pouring, such as the use of a plywood board to channel the grout. To minimize air entrapment, tremie-tube grouting should use a continuous grout flow. This requires that all the necessary grout for an individual connection be available at the outset of pouring and that a sufficiently large funnel be used. A funnel with a capacity of at least 4-quarts is recommended. The diameter of the tremie tube should be selected to adequately fit between the connectors and the duct.
2. Vent tubes should be used to vent air at the bedding layer. Proposed vent tubes should be tested during a trial batch to verify that the inner diameter is large enough for unrestricted flow of grout. A minimum diameter of ½ in. is recommended. Tubes placed through the bedding formwork should be located at the top of the bedding layer, rather than at mid-height, to ensure air is fully vented. Transparent vent tubes allow visual inspection of venting.
3. Residual water left in the connection after prewatering should be drained prior to grouting. Auxiliary water ports can be provided at the bottom of the bedding layer formwork for this purpose.
4. Trial batches should establish the required water volume and temperature to achieve the required flow for the expected on-site conditions and should obtain a strength satisfying the specification. Grouts should be monitored for segregation.
5. After mixing, grout should be filtered using a ½-in. mesh to ensure potential clumps are removed.
6. Grout should be placed in 2 to 3 lifts and tamped with a rod. Vibrators should be used with caution, as excessive agitation may entrap air.

7. To facilitate complete grouting of the bedding layer, no more than two shims per pile or column should be used. In addition, individual and total shim area can be limited. Shims should be placed at least 2 in. away from surface edges to help ensure grout completely surrounds the shims.
8. A bedding layer of 1.5 in. is adequate for conducting grouting operations.

4.2.5 Materials

4.2.5.1 Steel

ASTM A615 Grade 60 reinforcing steel was used for longitudinal reinforcement, stirrups, and spiral confinement in the beam and footing specimens, as well as for the epoxy-coated #9 bars used as connectors. ASTM A497 Grade 70 deformed welded wire fabric was used for mesh confinement. Bolts used in the CBC specimen were ASTM A193 B7 Grade 105 threaded rods. Yield and fracture strengths for connectors are listed in Table 4.4. See Section 3.4.2.1 for details of duct material.

Table 4.4 Connector Strengths (ksi)

Connector Size	**UT Laboratory Tests**	
	Yield	**Fracture**
#9 Rebar	66	110
1-in. Rod	90	110

4.2.5.2 Concrete

Similar to Phase 1, concrete used for Phase 2 specimens was the TxDOT Class C mix, with a minimum 28-day compressive strength of 3600 psi. See Section 3.2.4.3 for further details. Compressive strength curves are plotted in Figure 4.24 and show an average 28-day strength of 5400 psi.

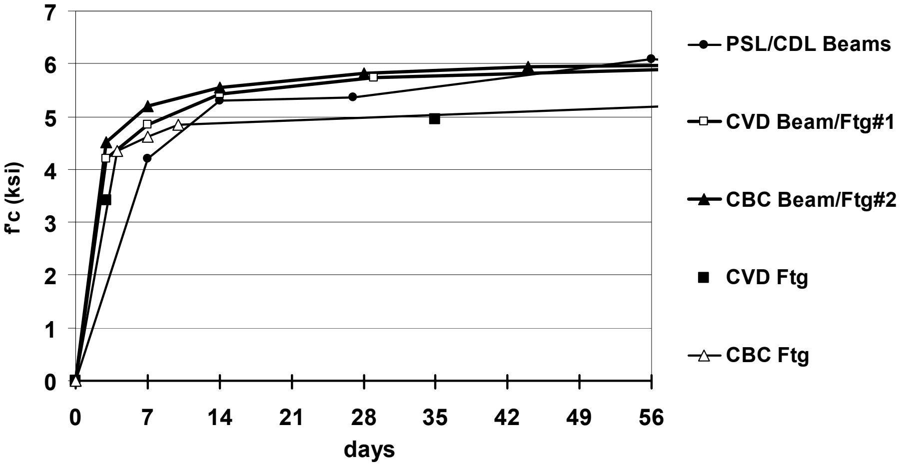


Figure 4.24 Concrete Compressive Strength Curves

4.2.5.3 Grout

The neat grouts used in Phase 1 were again used in Phase 2: EHF, Sika 212, and MF928. See Chapter 3 for further details. Figure 4.25 compares the modified grout cube strength for the different mixes.

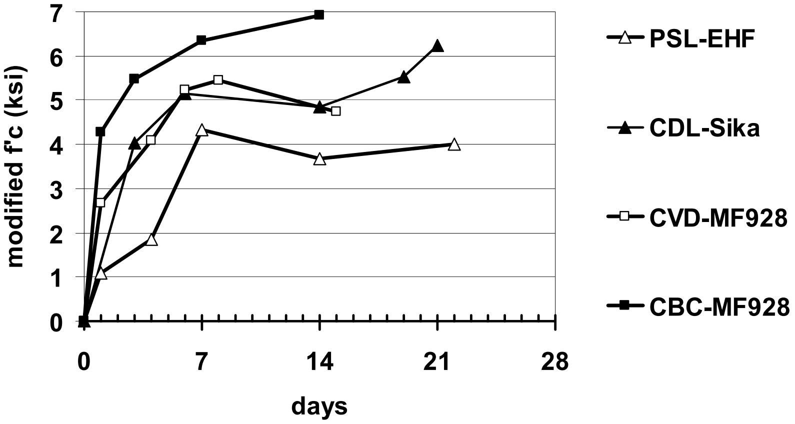


Figure 4.25 Grout Compressive Strength Curves

4.2.6 Instrumentation

The types of active and passive instrumentation were similar to those used in Phase 1, although a different data acquisition system was used. The acquisition system included a Hewlett Packard scanner and data acquisition software linked to a Microsoft Excel spreadsheet. The data acquisition system and front-end boxes are shown in Figure 4.26. Pressure transducers were used to measure the load applied by the vertical rams, and a load cell was used to measure the horizontal load. Figures 4.27 and 4.28 show the other active instrumentation used during testing.

Figure 4.26 Phase 2 Data Acquisition System

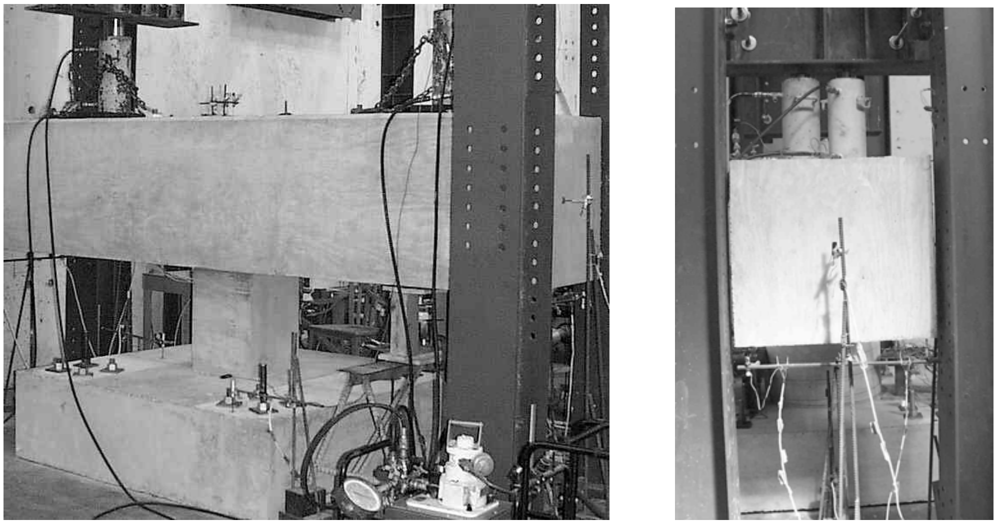


Figure 4.27 Phase 2 Instrumentation

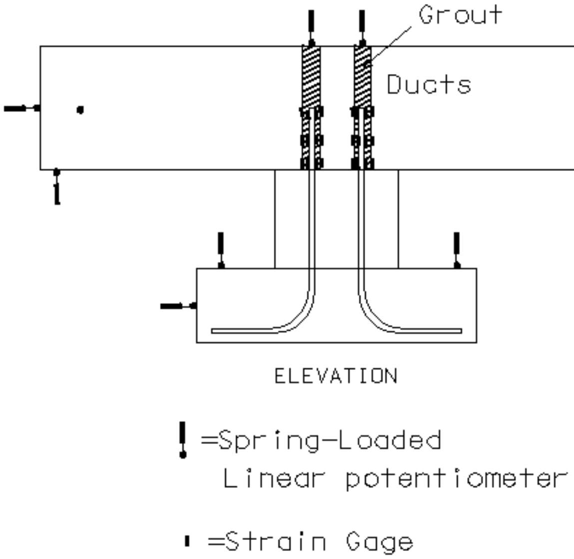
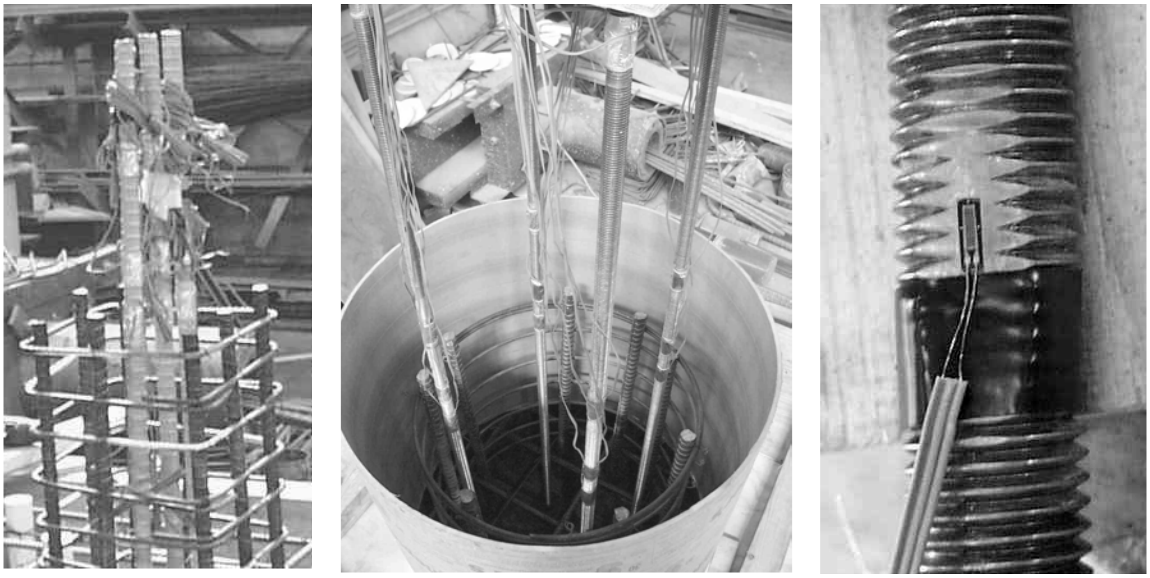


Figure 4.28 Instrumentation Schematic [4.1]

Connector strain measurements were important in determining both the connection behavior as well as the general behavior of the Phase 2 specimens. In addition to providing insight on the level of distress of individual connectors, strain gages near the column top were used to assess the adequacy of predicted response, including the strain distribution at various load levels as well as moment-curvature response. Strain was typically measured 1 in. above the bedding layer, at the midpoint along the embedment depth (7.5 in.), and, in some cases, 1 in. beneath the bedding layer (Figure 4.29a). Strain along connectors provided information on the anchorage of the connectors as well as the connection ductility. Strain was also measured 1 in. from the top of the CBC bolts (Figure 4.29b). Strain gages were the same as those used in Phase 1 (Section 3.2.5). An example of strain gage placement on a bolt is shown in Figure 4.29c.

For CVD and CBC specimens, strain gages were additionally placed on vertical ducts. For some cases, strain gages were placed in both the circumferential and seam directions. Duct gages were placed at 1 in., 7.5 in., and 15 in. above the bottom of the cap.



a. Gages on Bars b. Gages on Rods c. Close-up

Figure 4.29 Strain Gages

Cap and footing deflections were measured in both the horizontal and vertical directions using spring-loaded linear potentiometers. Cap deflections allowed load-deflection behavior to be determined. Footing deflections were measured to assess the fixity at the base of the pile or column. Relative displacement between the grout and concrete was also measured to investigate potential slip. Figures 4.27 and 4.28 show the various locations of the cap and footing linear potentiometers. Most active instrumentation provided reliable data.

The specimen was inspected for development and extension of cracks at each load increment. Cracks were typically measured at 20-kip intervals during loading, and crack patterns were marked on the specimen.

4.2.7 Test Setup

The test setup was designed to allow vertical and horizontal loading of the test specimens. Figure 4.30 shows a schematic of the test setup. As mentioned in Section 4.2.1.9, equal loads were applied at all rams, but rams were positioned at various locations to produce the intended axial load-moment combinations (i.e., eccentricities) at the connection. Because factored vertical loads for the cast-in-place column specimen approached 300 kips on each side of the connection, two 100-ton rams were required at each end of the cap. Rams rested on 1-in. neoprene bearing pads and reacted directly against W30x108 cross beams, which provided bearing along the cap width to accommodate longitudinal eccentricities as large as 9 in. Cross beams were supported by a pair of W30x108 girders. Holes for cross beams were drilled into the girders at intervals to allow loading at transverse eccentricities as large as 6.75 in. for rams applying load on both ends and 55.5 in. for a single ram applying load on the specimen. Vertical rams produced loads in individual columns that approached the 200-kip capacity at floor tie-down locations.

To represent the effect of wind, horizontal loads were applied simultaneously with vertical loads. Figure 4.31 shows the set up for the horizontal ram at the center of the cap end. A single 30-ton ram was used for the smaller horizontal loads.

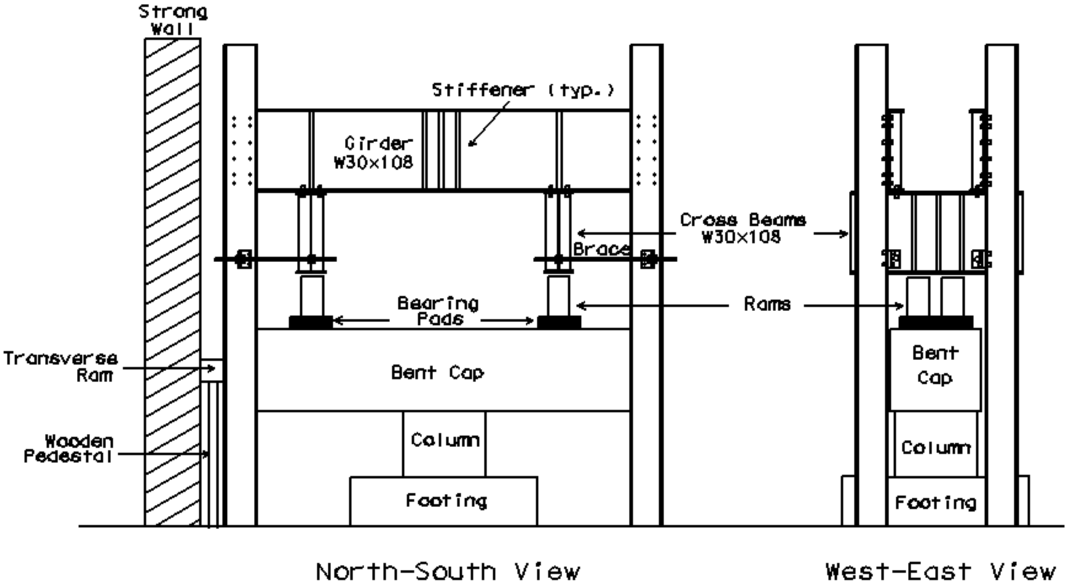


Figure 4.30 Schematic of Test Setup [4.1]

Figure 4.31 Horizontal Ram

4.2.8 Test Procedure

Testing was conducted by loading the vertical rams with a pneumatically-driven hydraulic pump. As shown in Figure 4.1, all hydraulic hoses were connected to a manifold at the pump, producing nearly equal pressure at each ram. Ram loads usually differed by no more than 2%. Air pressure was adjusted to apply a ram load (on each side) of approximately 10 kips per minute. Active channels were scanned automatically at 2-second intervals. Applied load versus deflection and applied load versus strain were plotted during tests to assess behavior, such as specimen cracking and bar yield. At each 20-kip increment, the load was held constant momentarily while the specimen was inspected and crack growth was marked. The total load on each side of the connection was marked on the specimen.

Figure 4.32 shows the general load sequence used in testing. For a particular combination of longitudinal and/or transverse eccentricity, the load sequence generally included 5 stages: 1) application of the service-level vertical load, 2) application and removal of a horizontal load, with the vertical load maintained, 3) increase of the vertical load to the factored level, 4) reapplication and removal of the horizontal load, and 5) removal of the vertical load. Horizontal loads represent transient loads such as wind and braking and were thus removed in stages 2 and 4. For simplicity, the factored horizontal load was applied for both stages 2 and 4, as the load level was relatively small. Four to six tests were conducted for each specimen.

4.3 SINGLE-LINE GROUT POCKET SPECIMEN ON PILE

4.3.1 Scope of Testing

Six tests were conducted using a single-line grout pocket specimen on a pile (PSL). Table 4.5 shows the test matrix for this series, including load-eccentricity combinations used for each test. Because of the uncertainty in response for the first specimen, eccentricity combinations were increased in relatively small increments. In addition, the first three tests, PSL1-PSL3, used only vertical loads. A horizontal load was applied for PSL4-PSL6 at the maximum service-level and factored-level loads. PSL6 also included a vertical load slightly larger than the factored level to further yield the connectors. PSL1-PSL4 can be considered proof tests for the connection. However, the eccentricity combinations of PSL5-PSL6 are larger than would normally be expected for a precast bent cap system, and are therefore considered failure eccentricity combinations. These were intended to investigate connection response under more severe loading, including bar yield.

4.3.2 Explanation of Tabulated Results for Phase 2 Tests

Phase 2 tests investigated the adequacy of force transfer through connections at both service and factored levels, as well as at failure. In addition to providing an understanding of behavioral characteristics, including anchorage of connectors, tests allowed the following aspects of strength, ductility, and stiffness to be determined: 1) capacity and failure modes of the connection, 2) inelastic response of connectors, 3) load-deflection response, and 4) moment-curvature response. Other important aspects of testing included determination of the level and extent of cracking of the specimen, bedding layer response, and influence of shims.

For each of the Phase 2 test series, results are presented using tables and plots. Tabulated results such as those shown in Table 4.5 include two categories: load-deflection response and connection response. The maximum applied load and corresponding deflection are listed for each load combination. Cap deflections were not intended to represent those of an actual bent cap system but are useful for comparison with analytical predictions. Connection response is expressed in terms of maximum tensile and compressive connector strains and the maximum crack widths at the top surface of the grout pocket (or at the cap top for CVD and CBC) at the service and factored levels. The "Delta" strain values show the difference between the recorded strain at the bedding layer and the calculated strain based on an analysis of the cross section for the actual axial load-moment combination.

The final column of a table lists a P_{max}/P_n ratio. P_{max}/P_n is the ratio of the maximum applied load to an equivalent axial load capacity at the top of the pile or column. The capacity is termed an "equivalent" axial load capacity because P_n reflects the effects of bending (either uniaxial or biaxial). P_{max}/P_n provides an estimate of how close the load effects approach the expected capacity. Except as noted otherwise in table footnotes, the capacity, P_n, is based on the Reciprocal Load Method (RLM), which accounts for biaxial bending [4.5]. For example, in Figure 4.34, P_n is based on an axial load-moment (P-M) strength interaction diagram in both directions for biaxial bending, using a strength reduction factor, ϕ, equal to 1.0. See footnotes in the tables for the RLM equation. Test-day pile or column strengths were used with actual connector yield strengths. Lines between the origin and data points for cases of uniaxial bending represent actual load paths. Other eccentricities were used in combination and are shown for comparison only.

Table 4.5 Single-Line Grout Pocket Test Matrix and Select Results

			Load-Deflection				Connection Response					
Test ID	Eccentricity (in)		Applied Load^A (kips)		Deflection^B (in)		Bar Strain^C (*10^{-6})				Crack width^D (*10^{-3} in)	Pmax/Pn^E
	Transv	Long	Vertical	Horiz	Vertical	Horiz	Tension	Delta	Comprsn.	Delta		
PSL1	3.75	0	94	0	0.13	n/a	(-10)	20	-190	70	5, 9	0.26 (C)
PSL2	3.75	3	94	0	0.14	n/a	(-10)	-80	-200	50	7, 13	0.37 (T)
PSL3	6.75	0	94	0	0.22	n/a	170	-490	-270	20	13, 13	0.51 (T)
PSL4	6.75	3	65, 94	20	0.27, 0.32	0.04	290, 450	-510	-270,-282	-20	16, 16	0.62 (T)
PSL5	6.75	6	65, 94	20	0.40, 0.45	0.05	1030, 1350	-260	-130, -154	-90	16, 16	0.95 (T)
PSL6	6.75	9	65,94,105	20	0.64,0.77,0.90	0.03	2120,4130,>7000	-220	(310, 430)	-200	16, 16	1.15 (T)

Footnotes

A. Applied load refers to load on each side of connection. Total load at pile top is twice the listed value.

B. Vertical deflections correspond to deflection at (maximum vertical load, maximum vertical load with horizontal load), where applicable

C. Bar strains correspond to maximum strains 1-in. above and below bedding. Multiple strain records were averaged. Compression is negative.
Delta strain value is actual strain minus prediction based on sectional analysis (for vertical load only).

D. Crack widths are maximum widths in grout pocket at service-level and factored-level loads

E. Pmax/Pn is the ratio of maximum applied load to an equivalent axial load capacity at the pile top, based on Reciprocal Load Method.
1/Pn=1/Pnxo + 1/Pnyo - 1/Po, where Pn=approximate value of ultimate load in biaxial bending; Pn>0.10Po
Pnxo=ultimate load when only eccentricity, ey, is present (ex=0)
Pnyo=ultimate load when only eccentricity, ey, is present (ex=0)
Po=ultimate load for concentrically loaded column
A value greater than 1.0 for PSL6 indicates the maximum applied load exceeded that predicted for biaxial bending.
C=connectors are not predicted to be in tension, T=some connectors are predicted to be in tension. Pmax for PSL6 is load at bar yield, 98 kips.

F. Test-day compressive strengths: concrete=5.9 ksi (column and cap), grout=4.0 ksi (modified)

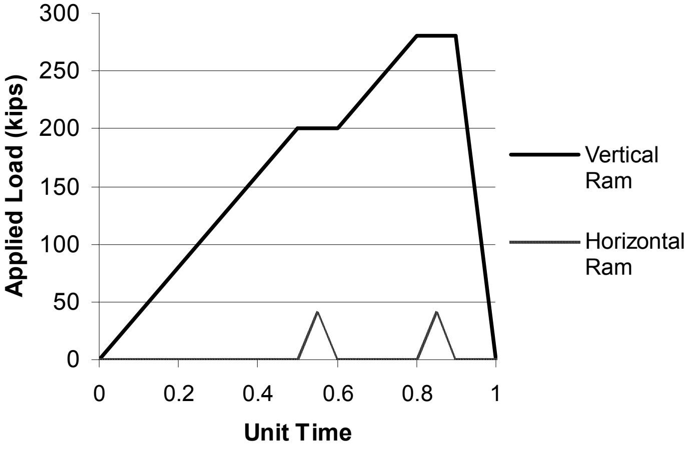
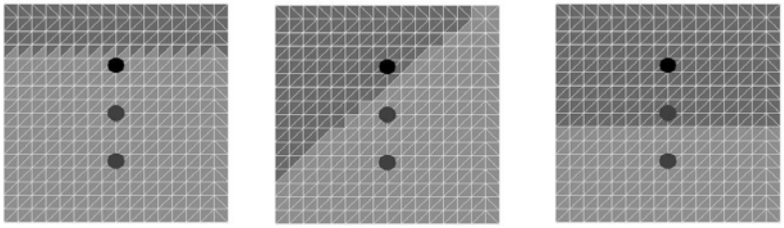
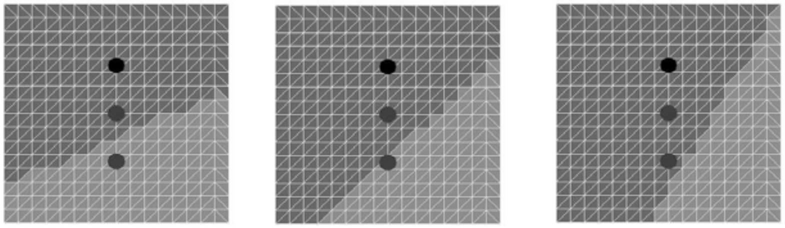


Figure 4.32 Phase 2 Load Sequence (Column Tests)



a. 3.75 in. Transverse
b. 3.75 in. Transverse;
3 in. Longitudinal
c. 6.75 in. Transverse



d. 6.75 in. Transverse;
3 in. Longitudinal
e. 6.75 in. Transverse;
6 in. Longitudinal
f. 6.75 in. Transverse;
9 in. Longitudinal

Figure 4.33 Predicted Neutral Axis Locations for PSL Tests

In addition to P-M plots, other plots include: 1) predicted vs. actual strain distribution over the column or pile cross section at the factored load, 2) predicted neutral axis locations at the factored load (Fig. 4.33), 3) predicted vs. actual load-deflection response, and 4) predicted vs. actual moment moment-curvature response.

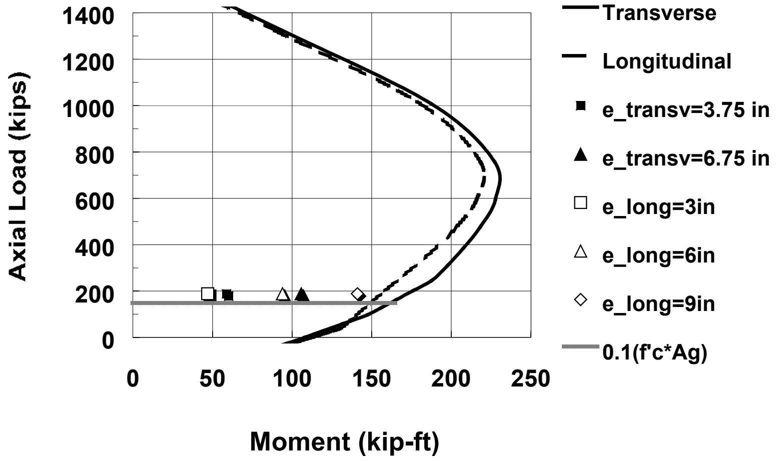


Figure 4.34 Axial Load-Moment Interaction Diagram for PSL 1/3 Connector Arrangement

4.3.3 Summary of Results

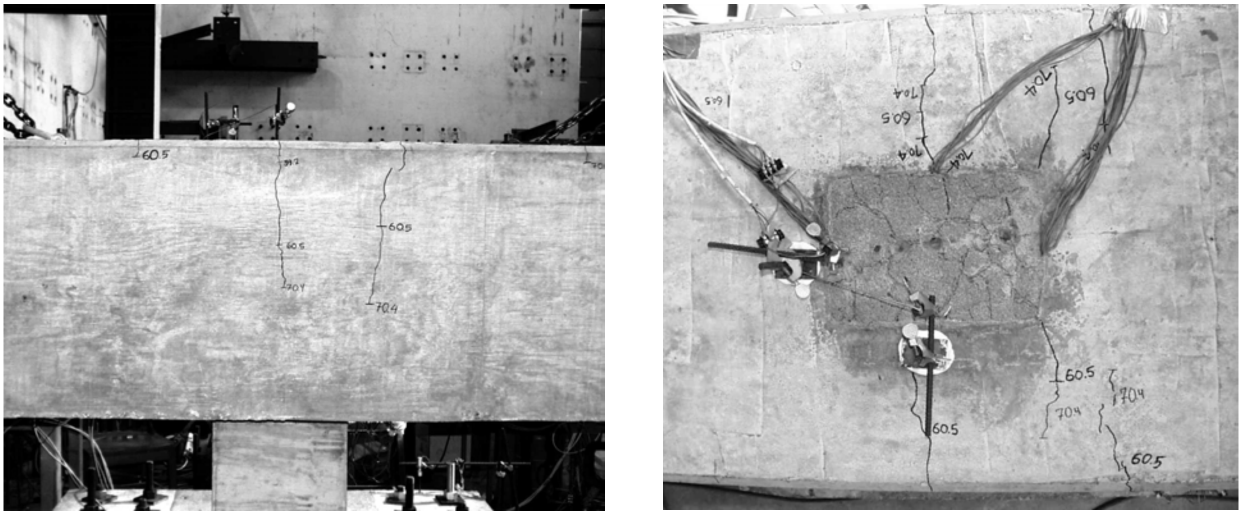
Table 4.5 lists select results for PSL tests. As expected from P_{max}/P_n ratios and Figure 4.34, PSL1-PSL4 tests exhibited minor levels of connection distress during proof tests. The maximum east connector strain was limited to 25% of bar yield, and thus bar pullout was not significantly challenged. Horizontal loads increased maximum tensile strains less than 10% of bar yield for proof tests and produced minor deflections. Flexural cracks developed through the grout pocket and widened under repeated loading. However, crack widths were no larger than 0.016 in. at the factored level and these cracks did not noticeably affect connection performance. Cracks as large as 0.01 in. also developed on the tension side of the pile during PSL3 and PSL4, indicating adequate transfer of forces through the connection and the importance of adequate anchorage of connectors into the pile. Cracks did not develop at the exposed portion of the bedding layer. Connector strains and curvatures compared reasonably well with predicted values.

The connection demonstrated excellent force transfer characteristics at the proof loads of PSL1-PSL4, as well as at the failure loads of PSL5 and PSL6. The load required for bar yield in PSL6 was 15% larger than predicted. Excellent anchorage of connectors within the grout pocket was evident as the east connector achieved yield at an embedment of $13d_b$ (concrete compressive strength of 5.9 ksi; modified grout cube strength of 4.0 ksi). Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. Slip at the grout-concrete interface was also negligible. Connector strains beyond the yield strain demonstrated suitable ductility. Pile cracks grew to more than 1/8 in. in failure tests, indicating bar yield in the pile as well. The fact that bar yield was evident above and below the bedding layer indicates the potential for spreading of inelastic action over a larger region than just the bedding layer. The development of strains as large as 50% of yield at the 7.5 in. location also suggests this.

The use of shims and the existence of a bedding layer had no discernable effect on response. In addition, the use of a grout with a compressive strength significantly lower than the cap concrete did not adversely affect performance.

4.3.4 PSL1

Using a 3.75-in. transverse eccentricity with loading to the factored level, PSL1 produced minor distress in the connection region, although significant flexural cracking of the cap occurred. Initial flexural cracking of the cap occurred over the pile in the negative moment region at the predicted cracking moment, which corresponded to an applied load of 60 kips. The applied load refers to the ram load on each side of the pile or column; the total load at the pile or column top is twice this value. Figure 4.35 shows the crack patterns on the cap side face and on the top surface in the grout pocket region at an applied load of 70 kips, which was close to the maximum service-level load of 65 kips. Side face cracks are shown to extend slightly more than half the cap depth at 70 kips. As shown in Figure 4.35b, one crack formed through the center of the grout pocket (where the middle connector is located) and joined preexisting grout pocket cracks, causing the grout pocket cracks to open up under loading. Other cracks emanated from the corners, where a stress concentration developed. Corner cracks were produced by flexural response of the specimen, unlike the Phase 1 corner cracks that were the result of bar pullout. Maximum crack widths were only 0.005 in.



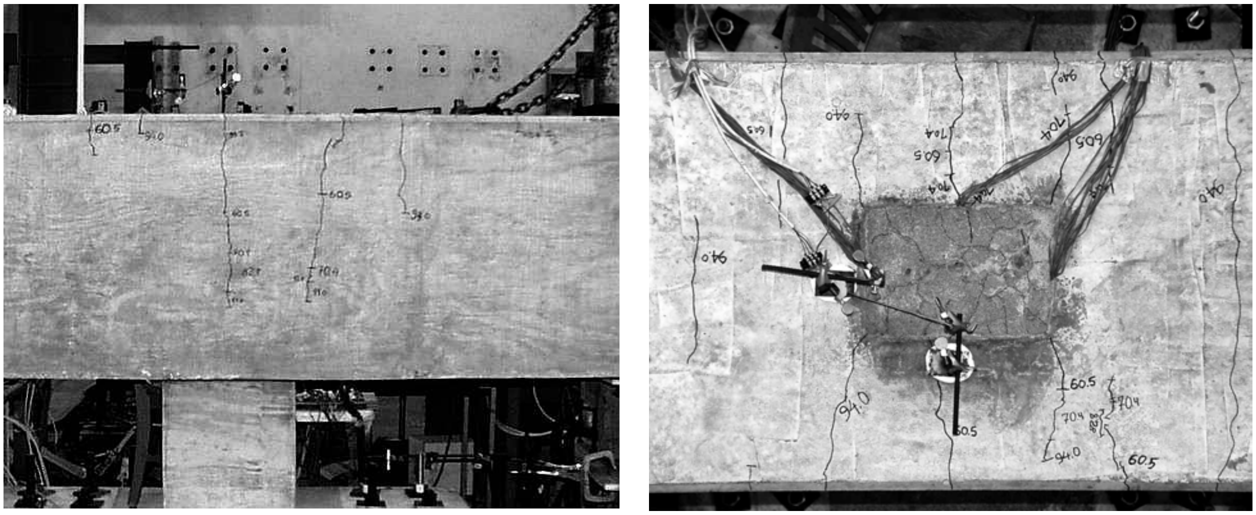
a. North Side Face *b. Cap Top*

Figure 4.35 PSL1 at Service-Level Load

Figure 4.36 shows the crack patterns at the factored load of 94 kips, which corresponded to 50% of the flexural capacity of the cap. New cracks appeared along the side face, with some extending ¾ of the cap depth (Figure 4.36a). Figure 4.36b exhibited additional flexural cracks on the cap top, including two through the pocket corners. Most crack widths were no larger than 0.009 in., although a few were 0.016 in. Figure 4.36b also shows one crack to the left of the grout pocket having a length approximately the same as the pocket width. Because longitudinal reinforcement cannot be placed through the central portion of the cap for a single-line grout pocket, crack opening was unrestricted. However, this crack did not form until the maximum factored load was applied.

Figure 4.37 shows the applied load-deflection response for the four corners of the specimen. The larger ram eccentricity was used on the west side. For ram loading at the cap centerline, SW and NW deflections were nearly the same, as were SE and NE deflections. A sudden reduction in cap stiffness on the west side is evident at 60 kips, due to formation of the first flexural cracks. The figure also shows that slip at the grout-concrete interface was negligible.

Figure 4.38 plots the applied load-strain response for PSL1 connectors. It should be noted that strain records were selected to accurately exhibit trends. However, strain levels plotted in graphs may differ from the strain values recorded in Table 4.5, which represent an average of strain gage measurements 1 in. above and below the bedding layer. Values in Table 4.5 thus represent the actual strain at the bedding level more accurately than individual records.



a. North Side Face *b. Cap Top*

Figure 4.36 PSL1 Crack Patterns at Factored Level

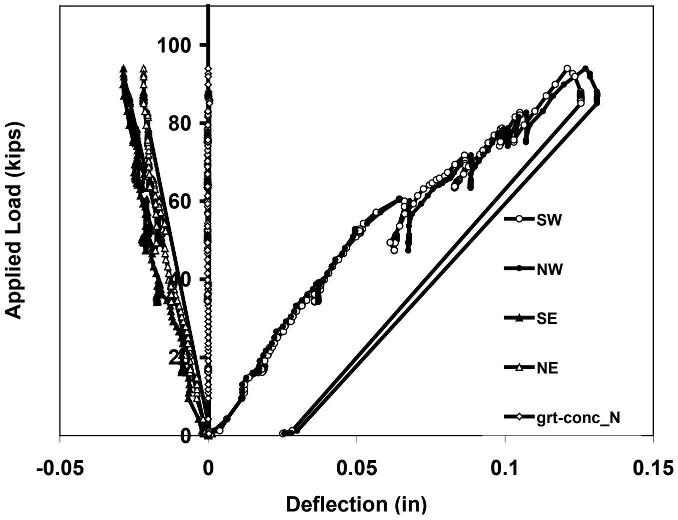


Figure 4.37 Load-Deflection Behavior for PSL1 Cap

Figure 4.38 reveals that all connectors were in compression and hence pullout of the bars was not significantly challenged. This pattern corresponds well with the predicted location of the neutral axis shown in Figure 4.33a (darker color corresponds to tension). Figure 4.39 plots the predicted and actual strain distribution across the pile top for cases of uniaxial bending, i.e., transverse eccentricity only (PSL1 and PSL3). Positive distance represents the east side of the pile. This plot shows a close comparison for both the magnitude of strain and curvature for PSL1.

Maximum strain levels were very small for all bar locations and at all depths along bars. The maximum compressive strain occurred at the west bar, which reached only ~10% of yield. Due to moment transfer, the east bar was strained to a smaller compressive strain, but did not develop tension. As was typical for most PSL tests, the connector strains at the 7.5-in. location were smaller than the 1-in. location. This indicates adequate development of bond along the bar in the grout.

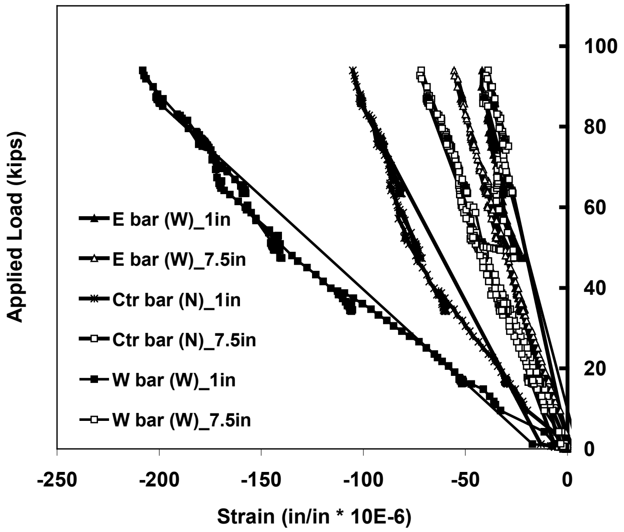


Figure 4.38 Load-Strain Behavior for PSL1 Connectors

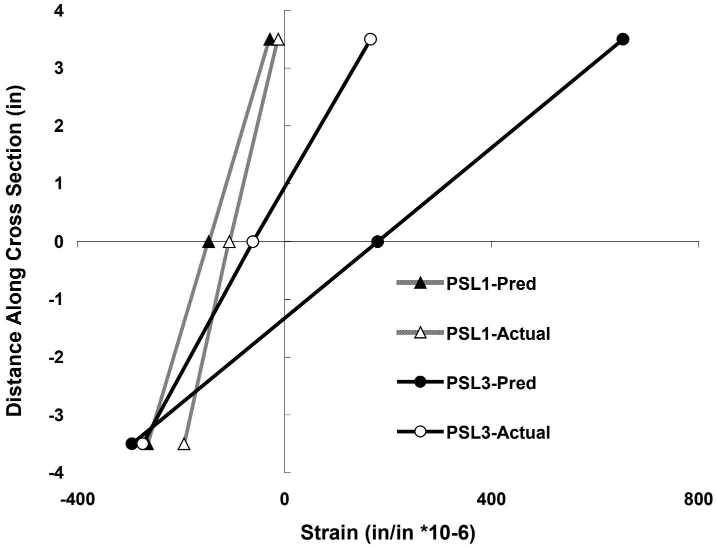


Figure 4.39 Predicted vs. Actual Strain Distribution for PSL1 and PSL3

4.3.5 PSL2

Using a slightly more severe eccentricity combination (3.75 in. transverse, 3.0 in. longitudinal), the PSL2 specimen again demonstrated minor connector distress, although cracking and deflections increased slightly. Figure 4.40 shows the load-deflection behavior. Because the cap had already cracked during PSL1, softening was not observed under loading. Instead, cracks re-opened under loading to widths slightly larger than duringPSL1. The application of combined eccentricities also caused biaxial bending, as shown by the larger deflection at the southwest and northeast ends. However, the maximum deflection was only slightly larger than that recorded duringPSL1. Cracks extended on the surface no more than about 3 in., but maximum crack widths in the grout pocket increased to 0.013 in.

Figure 4.41 demonstrates that small compressive strains again developed in the connectors. Although the maximum compressive strain in the west bar was nearly the same as during PSL1, connector strains in the other bars reduced considerably, confirming the change in orientation of the neutral axis associated with biaxial bending. Although a small tension in the east connector was expected based on the sectional analysis (Figure 4.33b), the neutral axis instead passed through the east connector, as evidenced by the near-zero strain. Although strains at the 7.5-in. location were normally smaller than at the bedding layer, occasionally a slightly larger strain was measured, as shown for the east bar.

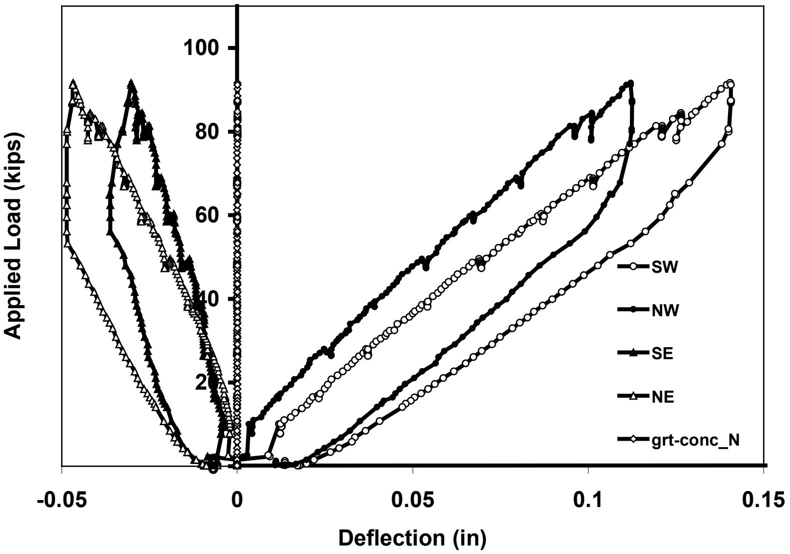


Figure 4.40 Load-Deflection Behavior for PSL2 Cap

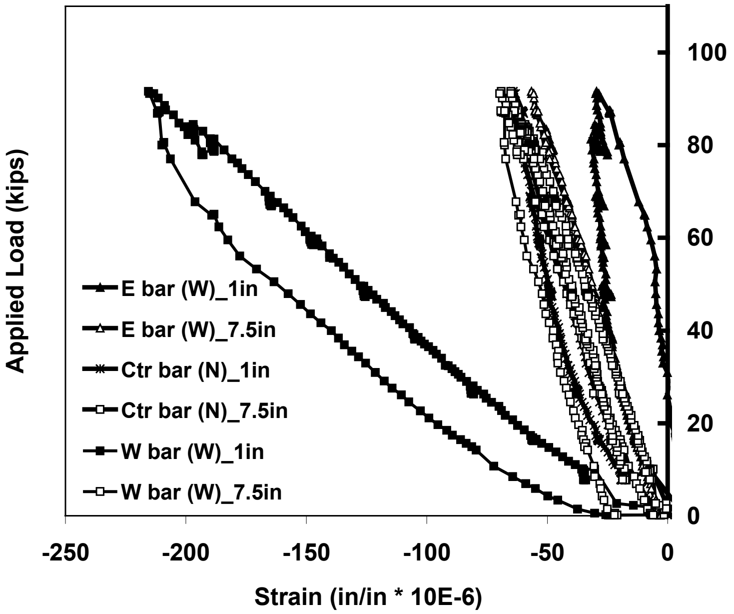
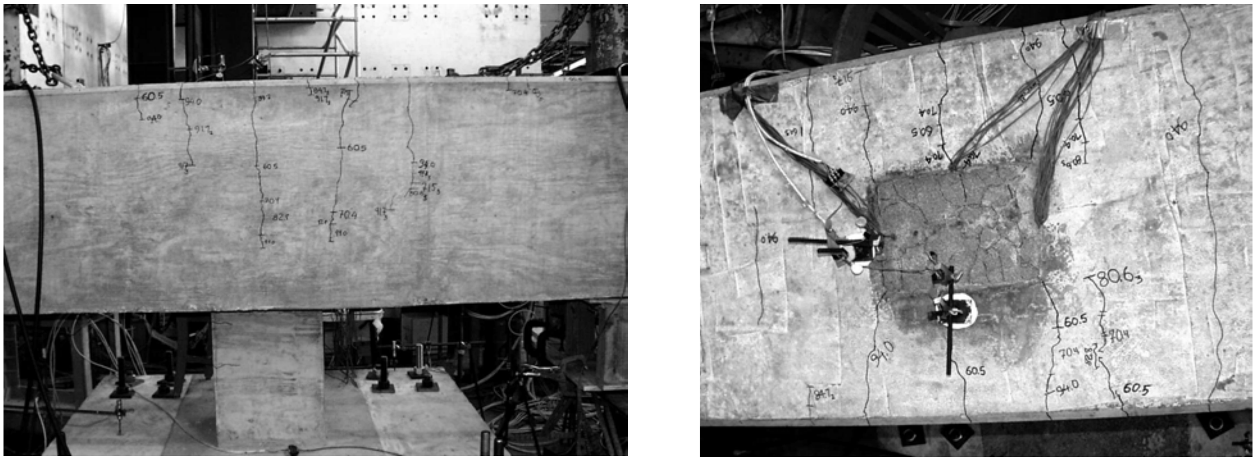


Figure 4.41 Load-Strain Behavior for PSL2 Connectors

4.3.6 PSL3

Uniaxial (transverse) bending was again used in PSL3, although at a considerably larger magnitude of 6.75 in. This value is considered an upper bound for a trestle pile bent in the transverse direction. As shown in Figure 4.42, existing cracks in the cap extended as the specimen was loaded to the factored level. Maximum crack widths were 0.013 in. in the grout pocket and 0.01 in. in the cap, respectively at service level.



a. North Side Face *b. Cap Top*

Figure 4.42 PSL3 Crack Patterns at Factored-Level Load

As expected for this eccentricity, tension developed on the east side of the specimen. At 92 kips, two cracks appeared on the pile, wrapping around the entire east side and several inches on the north and south sides, as shown in Figure 4.43 (a subscript “3” is used to designate PSL3). In addition, tension developed in the east bar, as shown in the load-strain plot in Figure 4.44. Figure 4.39, however, demonstrates that the level of tensile strain and curvature was smaller than expected. Figure 4.45 shows that the load-displacement response for the cap was essentially linear, but with the cap deflection considerably larger than during earlier tests.

Figure 4.43 PSL3 Pile Cracks (NE) at Factored-Level Load

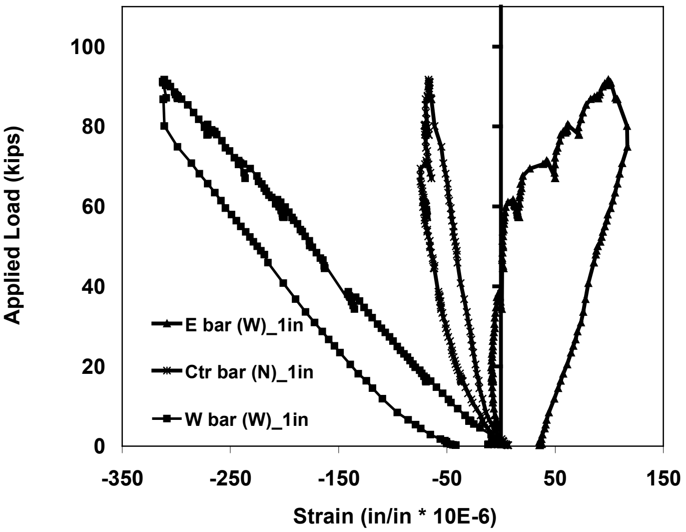


Figure 4.44 Load-Strain Behavior for PSL3 Connectors

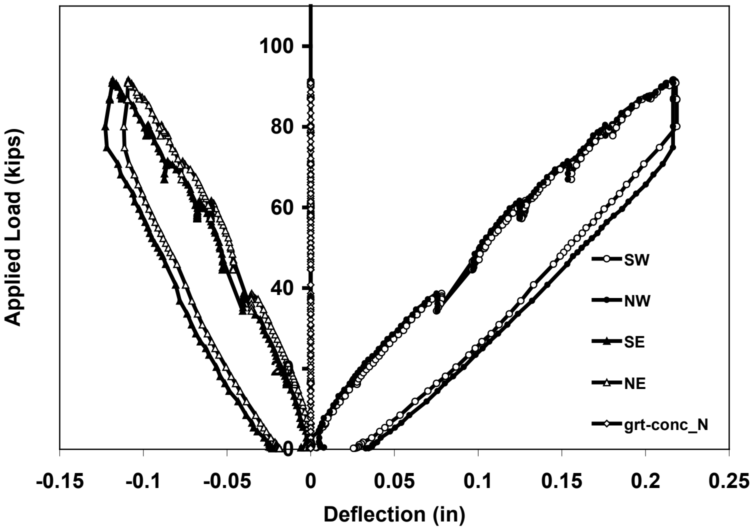


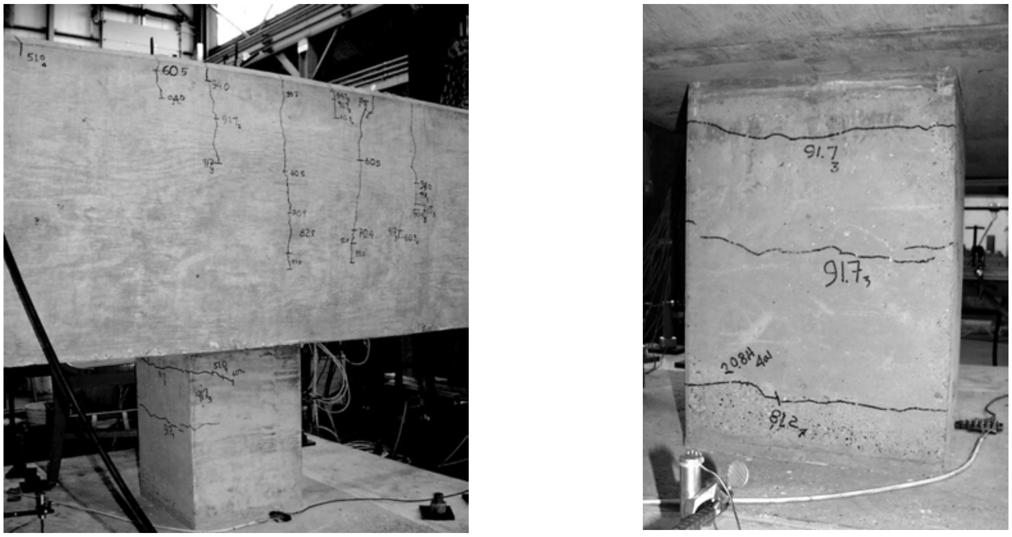
Figure 4.45 Load-Deflection Behavior for PSL3 Cap

4.3.7 PSL4

The PSL4 test used a combined eccentricity (6.75 in. transverse, 3.0 in. longitudinal) as well as a horizontal load at service and factored levels. Initially, the specimen was loaded to the factored level using only vertical loads. Then, the specimen was reloaded using the sequence shown in Figure 4.32. The discussion that follows accounts for both loading sequences.

Up to the service-level load, minor extensions of pile cracks occurred. At service level, a 20-kip horizontal load was applied, causing a new crack at the base of the pile on the east side and extensions of pile cracks. Figure 4.46b shows the new crack marked with an "H" (for horizontal load). Load-deflection and load-strain plots clearly show an increase in horizontal and vertical deflections as well as connector strains due to the effects of the horizontal load. A horizontal line is shown on the plots because the vertical load was maintained during the application of the horizontal load, although in some cases, the vertical load decreased slightly. Horizontal load versus cap end deflection is included in the plot of Figure 4.47 and exhibits reasonable horizontal frame stiffness. Figure 4.48 shows that connector strains increased significantly due to horizontal loading, especially on the east (tension) side, although maximum levels were still less than 15% of yield. No change in grout pocket surface cracking was observed.

After the lateral load was removed, most of the connector strains and cap deflections were recovered. Vertical loads were then increased to the factored level. Despite pile cracking due to the horizontal load, the frame stiffness under vertical loading appeared to be nearly the same as upon initial PSL4 loading. When the lateral load was reapplied, an increase in response similar to that observed at the service level developed. Figure 4.46 shows the crack patterns for the cap and pile. The crack width at the northeast corner of the pile top increased to 0.01 in. Although the east connector strain increased by 2/3, this amounted to an increase of only 160 microstrain and a total strain that was 20% of yield.



a. North Side Face *b. East Side of Pile*

Figure 4.46 PSL4 Crack Patterns at Factored-Level Load

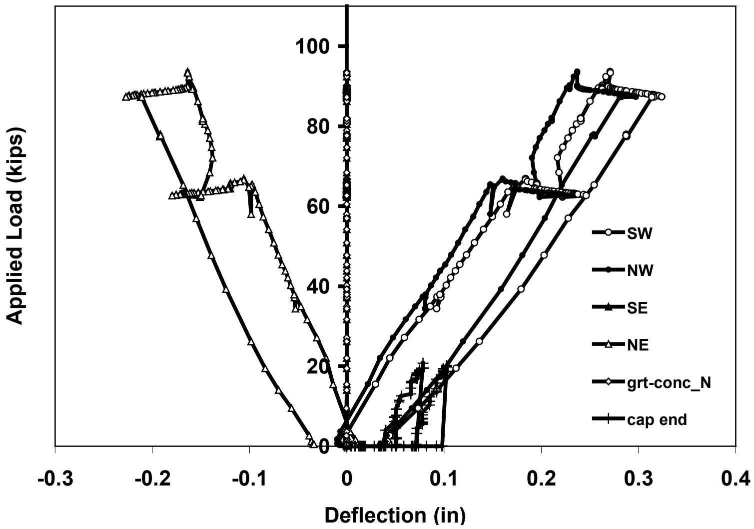


Figure 4.47 Load-Deflection Behavior for PSL4 Cap

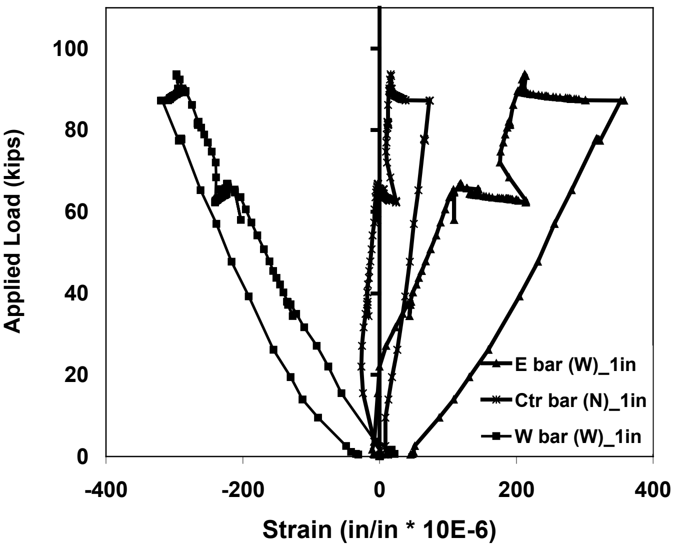


Figure 4.48 Load-Strain Behavior for PSL4 Connectors

4.3.8 PSL5

The PSL5 test used a failure eccentricity combination of 6.75 in. transverse and 6.0 in. longitudinal. For comparison, loading was still applied in stages. As shown in Figure 4.49, horizontal and vertical cap deflections increased significantly over PSL4 deflections. Figure 4.50 shows that the east and center connectors were strained much more than during PSL4. This corresponds to the pattern prediction in Figure 4.33e. Figure 4.51 shows crack extensions at the north side of the pile at 65 kips. Pile cracks changed orientation (downward) due to the lateral load (see Figure 4.52b).

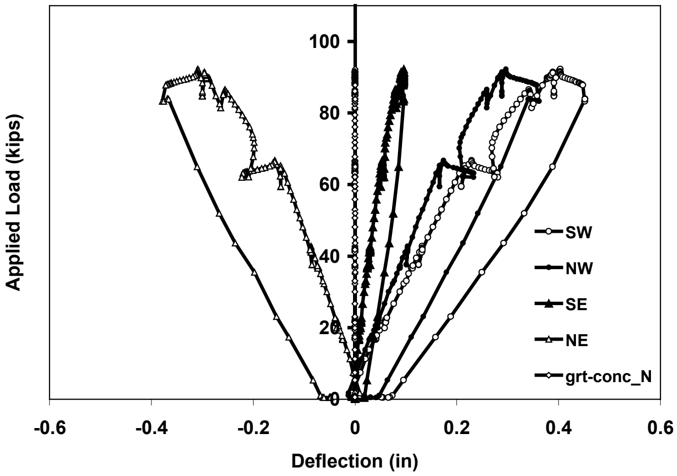


Figure 4.49 Load-Deflection Behavior for PSL5 Cap

Near 94 kips, the load-deflection plot exhibits a softening of the system. This corresponded to the extension, opening, and new formation of cracks at 87 kips. Figure 4.51a shows a new crack near the interface between the pile top and bedding layer. This crack wrapped around the north and east faces of the pile and opened to a large 0.02-in. width. Strains increased noticeably upon loading to the factored level, and after addition of the horizontal load, tensile strains reached 60% of yield (Table 4.5). Despite the large bar strains, grout pocket cracks at the surface did not appear to open beyond pre-existing crack widths. Overall connection response demonstrated the adequacy of the grout pocket connection in transferring forces associated with unrealistically large eccentricities.

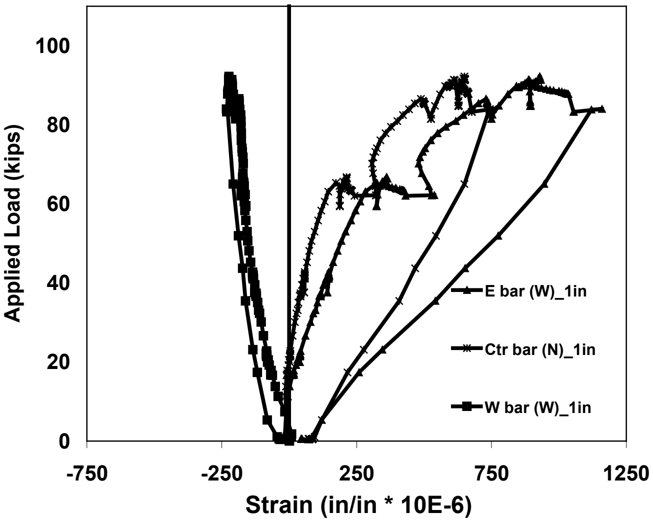
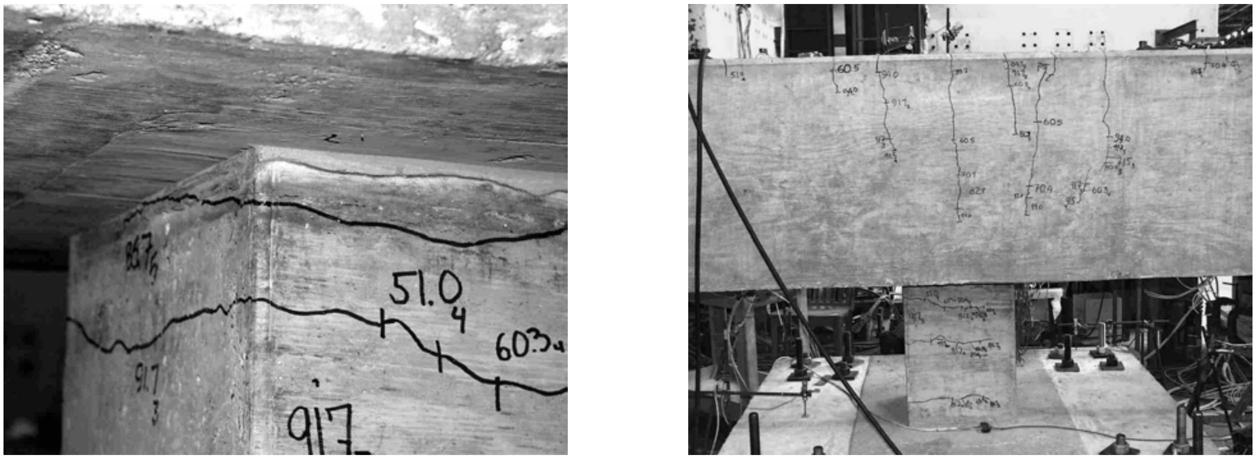


Figure 4.50 Load-Strain Behavior for PSL5 Connectors



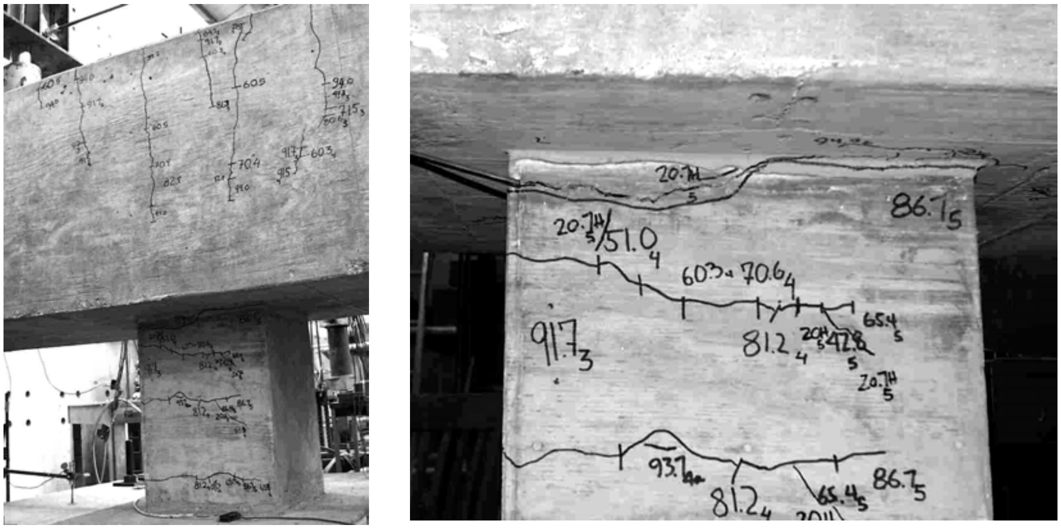
a. NE Side of Pile *b. North Side Face*

Figure 4.51 PSL5 Crack Patterns at Factored-Level Load

4.3.9 PSL6

To further challenge the connection, PSL6 used a failure eccentricity combination of 6.75 in. transverse and 9.0 in. longitudinal. At a load of 65 kips vertical and 20 kips horizontal (service level for proof tests), cracks extended across the pile as shown in Figure 4.52a. In addition, cracks appeared at the bedding layer on most sides of the pile. Figure 4.53 shows that the west bar was in tension and that the east bar strain reached 60% of yield.

Similar to PSL5 response, softening of the system occurred upon loading to the factored level and corresponded to extensions, large openings, and new formation of cracks. Figure 4.52b shows the crack opening at the northeast corner of the pile to be over 1/8 in. Multiple cracks were observed at the pile top and bedding layer (Figure 4.54a), suggesting yield of one or more connectors in the pile. In addition, a large crack extended on the south side of the beam (Figure 4.54b). After addition of the horizontal load, vertical displacements increased to 0.8 in. (Figure 4.53) and the east connector just reached yield, although significant variation in strain records indicated bending of the bar.



a. At 70 kips *b. At 94 kips*

Figure 4.52 PSL6 Pile Crack Patterns on North Side

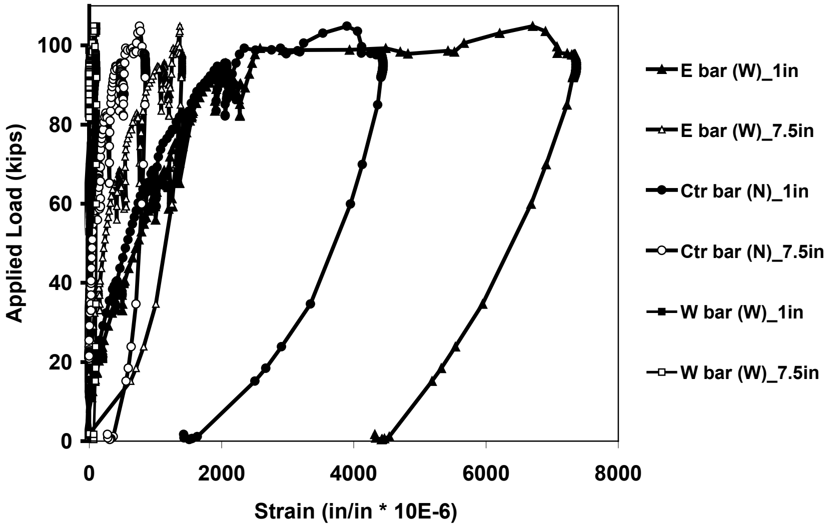
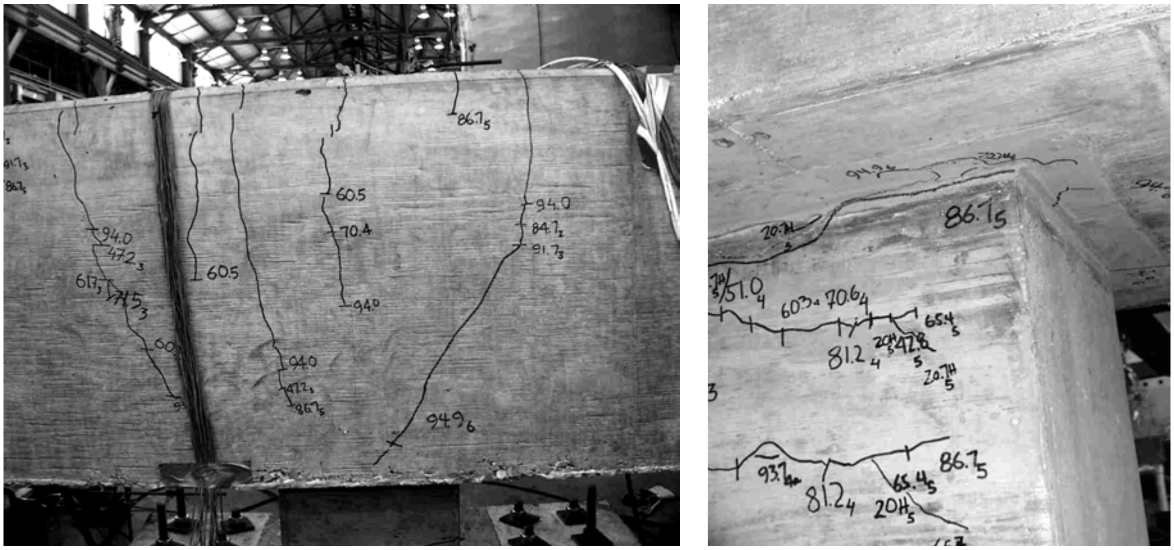


Figure 4.53 Load-Strain Behavior for PSL6 Connectors

In an attempt to investigate connection response for further bar yield and strain hardening, the vertical load was increased. At 99 kips, the east bar strains increased significantly at the bedding layer, as shown in Figure 4.53. Figure 4.55 shows that this was accompanied by a noticeable increase in vertical deflection. As the ram was pumped further, the east bar entered the strain hardening regime, although the east bar strain at the 7.5-in location was only ~50% of yield. At 105 kips, the specimen was unloaded. No change in grout pocket surface cracking was observed, and no additional cracks formed around the pocket.



a. South Side Face *b. NW Side of Pile*

Figure 4.54 PSL6 Crack Patterns

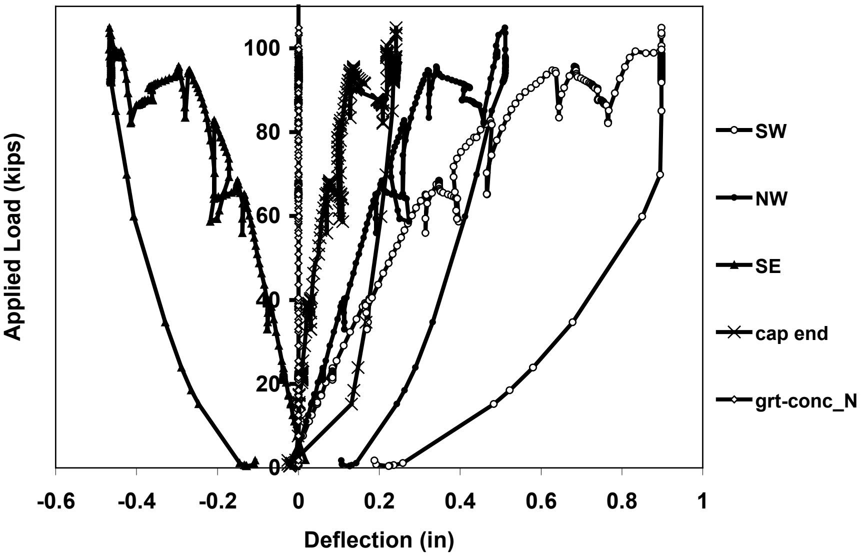


Figure 4.55 Load-Deflection Behavior for PSL6 Cap

The connection demonstrated excellent force transfer characteristics. Excellent anchorage of connectors within the grout pocket was evident as the east connector achieved yield at an embedment of 13d_b. Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. The development of connector strain beyond yield demonstrated suitable ductility. Furthermore, the fact that bar yield was evident above and below the bedding layer indicates the potential for spreading of inelastic action over a larger region than just the bedding layer. The development of strains as large as 50% of yield at the 7.5 in. location confirms this. The use of shims and the existence of a bedding layer had no discernable effect on response.

4.4 DOUBLE-LINE GROUT POCKET SPECIMEN ON CAST-IN-PLACE COLUMN

4.4.1 Scope of Testing

Four tests were conducted using a double-line grout pocket connection with a cast-in-place column. Table 4.6 shows the test matrix for the CDL test series and lists the load-eccentricity combinations used for each test. The first specimen, CDL1, used only vertical loads to investigate response at service and factored levels. After uniaxial behavior was observed in CDL1, combined eccentricities and horizontal loading were applied in CDL2 and CDL3. Based on PSL test results, relatively large increments of eccentricity were used between tests. The first three tests are considered proof tests. CDL4 used a 55.5-in. transverse eccentricity to yield connectors.

4.4.2 Summary of Results

Table 4.6 lists select results for CDL tests, which showed several similarities to PSL proof tests. As expected from P_{max}/P_n ratios and Figure 4.56, CDL1-CDL3 specimens exhibited minor levels of connection distress during proof tests. The maximum tensile strain of the NE connector was limited to 20% of bar yield. Thus, bar pullout was not significantly challenged. Horizontal loads increased maximum tensile strains less than 10% of bar yield and produced minor deflections. Flexural cracks developed through the grout pockets and widened under repeated loading. However, crack widths were no larger than 0.016 in. at the factored level and these cracks did not noticeably affect connection performance. Cracks also developed on the tension side of the column during CDL3, indicating adequate transfer of forces through the connection. Vertical cracks formed at the exposed portion of the bedding layer at service and factored levels, but widths were less than 0.002 in.

The failure eccentricity used in CDL4 produced much more severe response. The east bars yielded at both the bedding layer and the 7.5-in. depth. However, no evidence of significant bar slip was observed, nor any change in surface cracking of the grout pocket. Slip at the grout-concrete interface was negligible for all tests. This indicates that adequate bond developed between the connectors and grout, as well as between the grout and concrete. However, under loading at the 55.5-in. eccentricity, a ¼-in. gap gradually opened at the east side of the bedding layer. In addition, vertical cracks did form in the column due to splitting stresses associated with connector anchorage.

Comparison of the load-deflection response for CDL4 to a prediction based on a frame analysis using a rigid connection demonstrated the stiffness of the specimen to be smaller by a factor of approximately 2 to 2.4, due primarily to the small number of connectors. A moment-curvature plot showed a close match in slopes for the CDL specimen and a prediction based on a sectional analysis. This comparison indicated that response of a precast bent cap system might be reasonably predicted from analysis if the actual number and location of connectors were accounted for in the determination of joint stiffness. Strain values listed in Table 4.7 were reasonably close to predictions prior to bar yield.

The connection demonstrated outstanding force transfer characteristics. Excellent anchorage of connectors within the grout pocket was evident as the NE and SE connectors achieved yield at an embedment of $13d_b$. Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. Excellent ductility was exhibited not only by the achievement of bar yield and strain hardening, but by the significant spreading of yield along at least 7.5 in. of the connector length. The presence of shims and a bedding layer had no discernable effect on response. In addition, the use of a grout with a compressive strength significantly lower than the cap concrete did not affect performance.

Table 4.6 Double-Line Grout Pocket Test Matrix and Select Results

			Load-Deflection				Connection					
Test ID	Eccentricity (in)		Applied Load[A] (kips)		Deflection[B] (in)		Bar Strain[C] ($*10^{-6}$)				Crack width[D] ($*10^{-3}$ in)	Pmax/Pn[E]
	Transv	Long	Vertical	Horizontal	Vertical	Horizontal	Tension	Delta	Comprsn.	Delta		
CDL1	4.25	0	285	0	0.19	n/a	(-50)	50	-350	-90	7, 9	0.24 (C)
CDL2	4.25	6	200, 285	40	0.26, 0.29	0.02	150, 200	10	-640, -670	-240	13, 13	0.37 (T)
CDL3	7.25	6	200, 285	40	0.38, 0.41	0.03	230, 230	310	-750, -800	-250	13, 16	0.51 (T)
CDL4	55.5	0	83	0	0.88	n/a	yield	n/a	770	190	16	0.88[E] (T)

Footnotes

A. Applied load refers to load on each side of connection. Total load at column top is twice the listed value.

B. Vertical deflections correspond to deflection at (maximum vertical load, maximum vertical load with horizontal load), where applicable

C. Bar strains correspond to maximum strains 1-in. above and below bedding. Multiple strain records were averaged.

Compression is negative. Delta values are actual strain minus prediction based on sectional analysis (for vertical load only).

Due to loss of vertical load during application of horizontal load, maximum tensile strain may be as much as 170 microstrain low.

D. Crack widths are maximum widths at service and factored levels. Cracks correspond to the grout pocket.

E. Pmax/Pn is the ratio of maximum applied load to an equivalent axial load capacity at the column top, based on the Reciprocal Load Method. Sectional analysis for P-M curve is based on 2/2 connectors spaced at 12 in. each direction.

1/Pn=1/Pnxo + 1/Pnyo - 1/Po, where Pn=approximate value of ultimate load in biaxial bending; Pn>0.10Po

Pnxo=ultimate load when only eccentricity, ey, is present (ex=0)

Pnyo=ultimate load when only eccentricity, ey, is present (ex=0)

Po=ultimate load for concentrically loaded column

Actual locations varied +/-1/2 in. C=connectors are not predicted to be in tension, T=some connectors are predicted to be in tension

Pmax for Test 4 is the load at bar yield: 61 kips for CDL4. Pmax/Pn is 61/69=0.88, scaled from P-M plot. The maximum applied load was 83 kips.

F. Test-day compressive strengths: CDL: concrete=6.0 ksi (column and cap), grout=5.1 ksi (modified)

Table 4.7 Cast-in-place Column Bent Test Matrix and Select Result

			Load-Deflection				Connection					
Test ID	Eccentricity (in)		Applied Load^A (kips)		Deflection^B (in)		Bar Strain^C (*10^{-6})				Crack width^D (*10^{-3} in)	Pmax/Pn^E
	Transv	Long	Vertical	Horizontal	Vertical	Horizontal	Tension	Delta	Comprsn.	Delta		
CDL1	4.25	0	285	0	0.19	n/a	(-50)	50	-350	-90	7, 9	0.24 (C)
CDL2	4.25	6	200, 285	40	0.26, 0.29	0.02	150, 200	10	-640, -670	-240	13, 13	0.37 (T)
CDL3	7.25	6	200, 285	40	0.38, 0.41	0.03	230, 230	-310	-750, -800	-250	13, 16	0.51 (T)
CDL4	55.5	0	83	0	0.88	n/a	yield	n/a	770	190	16	0.88^E (T)
CVD1	4.25	0	285	0	0.20	n/a	(-50)	50	-170	-120	7, 9	0.28 (C)
CVD2	4.25	6	200, 285	40^F	0.29	0.03	-310	210	-270	0	7, 9	0.46 (T)
CVD3	7.25	6	200, 285	40	0.34, 0.38	0.03	520, 520	-90	-310, -320	260	7, 9	0.62 (T)
CVD4	55.5	0	94	0	0.83	n/a	yield	n/a	(650)	-1280	13	1.06^E (T)
CBC1	4.25	0	285	0	0.21	n/a	60	170	-150	20	7, 9	0.28 (C)
CBC2	4.25	6	200, 285	40	0.23, 0.27	0.02	340, 380	250	-240, -270	0	7, 9	0.46 (T)
CBC3	7.25	6	200, 285	40	0.33, 0.36	0.02	560, 570	-100	-300, -300	280	9, 13	0.62 (T)
CBC4	55.5	0	87	0	0.77	n/a	yield	n/a	(-1120)	-200	n/a	1.00^E (T)

A. Applied load refers to load on each side of connection. Total load at column top is twice the listed value.

B. Vertical deflections correspond to deflection at (maximum vertical load, maximum vertical load with horizontal load), where applicable

C. Bar strains correspond to maximum strains 1-in. above and below bedding. Multiple strain records were averaged.
Compression is negative. Delta values are actual strain minus prediction based on sectional analysis (for vertical load only).
Due to loss of vertical load during application of horizontal load, maximum tensile strain may be as much as 170 microstrain low.

D. Crack widths are maximum widths at service/factored levels and correspond to grout pocket for CDL, cap top for CVD/CBC.

E. Pmax/Pn is the ratio of maximum applied load to an equivalent axial load capacity at the column top. Pn is based on the Reciprocal Load Method. Sectional analyses for P-M curves are based on 2/2 connector arrangement at 12 in. each direction.
Actual locations varied +/-1/2 in. C=connectors are not predicted to be in tension, T=some connectors are predicted to be in tension
Pmax for Test 4 is the load at bar yield: 61 kips for CDL4, 71 kips for CVD4, and 78 kips for CBC4. Pmax/Pn is scaled from P-M plot.

F. Horizontal load applied at service level only

G. Test-day compressive strengths: CDL: concrete=6.0 ksi (column and cap), grout=5.1 ksi (modified)
CVD: concrete=4.9 ksi (column), 6.0 ksi (cap); grout=5.2 ksi (mod) CBC: concr=4.8 ksi (col), 6.0 ksi (cap); grout=6.1 ksi (mod)

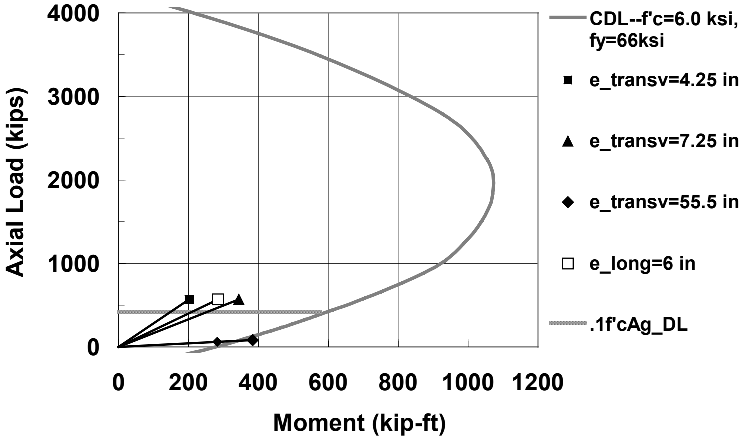
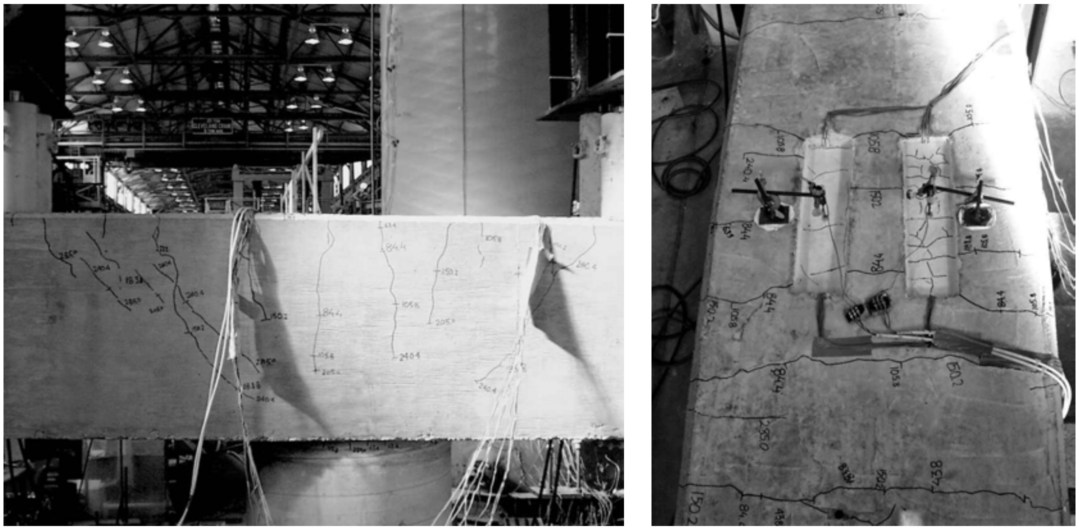


Figure 4.56 Axial Load-Moment Interaction Diagram for CDL 2/2 Connector Arrangement

4.4.3 CDL1

CDL1 produced minor distress in the connection region, although significant flexural cracking of the cap occurred. First flexural cracking of the cap occurred at the cracking load of 60 kips. Figure 4.57 shows the crack patterns on the cap side face and on the top surface in the grout pocket region at the factored level load of 285 kips (each side), corresponding to 2/3 of the flexural and shear capacity of the cap specimen. Thin cracks less than 0.005 in. wide appeared on the top surfaces of the grout pockets at 22 kips (Figure 4.57b). These cracks developed due to the weak, paste-like surface that formed because of grout segregation, but were not of concern structurally. As the load increased to 84 kips, cracks appeared across grout pockets, at the pocket corners, as well as on the sides. At 183 kips, shear cracks first appeared on the side face. At the maximum service load of 200 kips, flexural cracks extended to ¾ of the cap depth. However, crack widths on the beam and grout pocket surfaces were no larger than 0.009 in. and 0.007 in., respectively.



a. South Sidewall *b. Cap Top*

Figure 4.57 CDL1 Crack Patterns at Factored-Level Load

As the load was increased to the factored level, numerous additional shear cracks appeared on the side face of the cap. Existing flexural cracks widened to as large as 0.013 in. on the sides and 0.009 in. in the pockets. Figure 4.58 shows hairline cracks (<0.002 in.) at the bedding layer, most of which appeared as the load was increased beyond the service level. These cracks formed due to the Poisson effect and indicate the transfer of large compressive forces to the column. Such cracks do not pose strength or durability concerns.

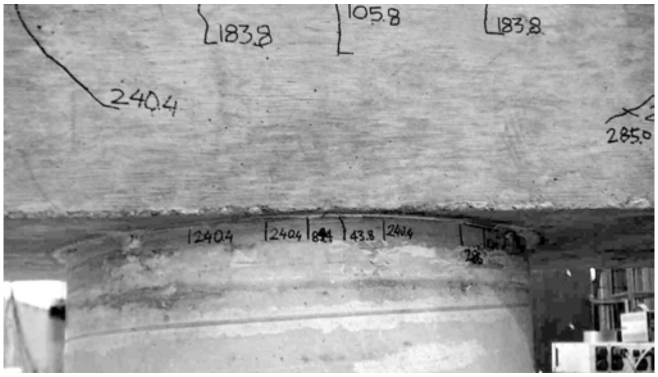


Figure 4.58 CDL1 Bedding Layer Cracks at Factored-Level Load

Figure 4.59 plots load-deflection for CDL1 and shows a stiffer system response compared to PSL1, due to differences in number and location of connectors as well as in longitudinal reinforcement used in the cap. A large reduction in stiffness is evident at the cracking load of 60 kips, and the subsequent decrease in stiffness corresponded to crack growth. Figure 4.59 also shows that slip at the grout-concrete interface was negligible.

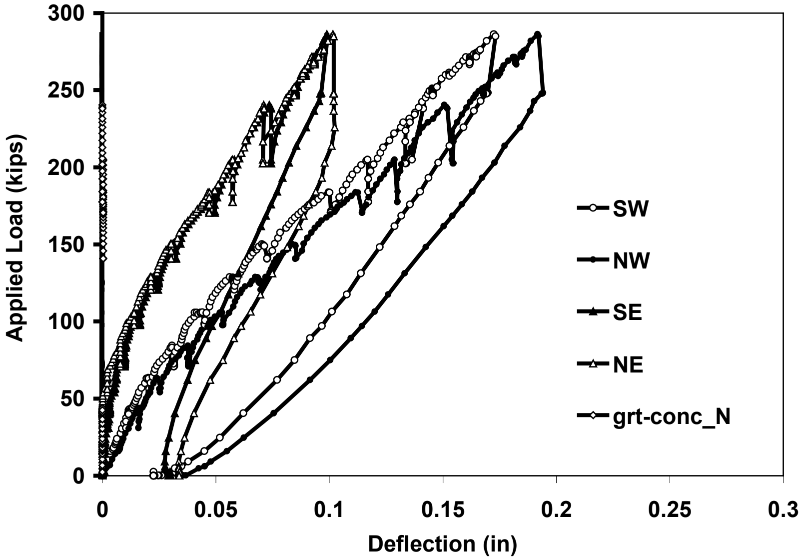
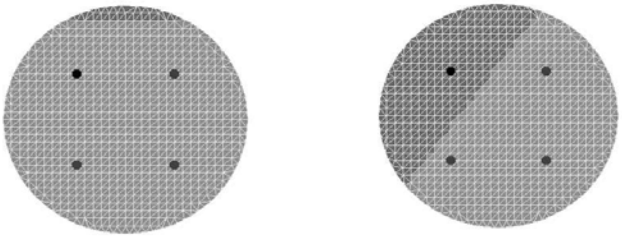


Figure 4.59 Load-Deflection Behavior for CDL1 Cap

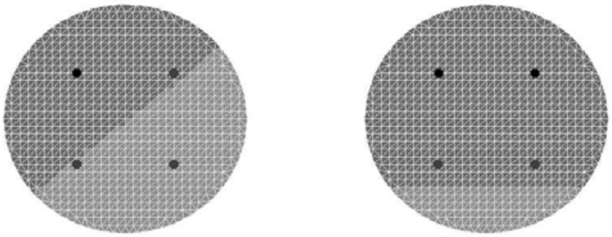
Despite the significant flexural and shear response of the cap, instrument records indicated that the connectors sustained a low level of strain, with a maximum strain of less than 20% of yield compression). Figure 4.61 reveals that all connectors were in compression, as expected from Figure 4.60a. Figure 4.61 exhibits some differences in strain between north and south bars (even when bar bending is accounted for), despite the use of only transverse eccentricity. The presence of a bedding layer void may have resulted in an increase in the SW connector strain, although this effect should be minor given a large bearing area for the small eccentricity. Nevertheless, the general strain distribution matched predictions reasonably well, as shown in Table 4.6. Smaller strains developed at the 7.5-in. location, indicating adequate development of bond along the bar.

No evidence was found indicating that flexural cracks through the grout pocket or the presence of shims at the bedding layer adversely affected connection performance.



a. 4.25 in. Transverse

b. 4.25 in. Transverse; 6 in. Longitudinal



c. 7.25 in. Transverse; 6 in. Longitudinal

d. 55.25 in. Transverse

Figure 4.60 Predicted Neutral Axis Locations for Column Tests

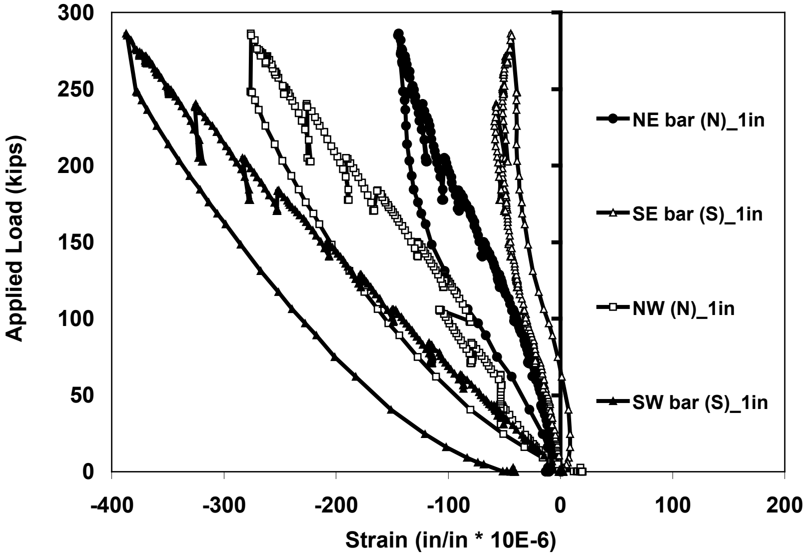


Figure 4.61 Load-Strain Behavior for CDL1 Connectors

4.4.4 CDL2

CDL2 used an eccentricity combination of 4.25 in. transverse and 6.0 in longitudinal. Figure 4.62 exhibits the nearly linear load-deflection behavior for the cracked specimen. Upon reloading, cracks in the grout pockets and at the cap top opened to slightly larger widths of 0.013 in. As the vertical load was increased to service and factored levels, new flexural and shear cracks appeared throughout the specimen, as shown in Figure 4.63. In addition, a crack appeared at the base of the column and extended into the footing at the maximum service-level load.

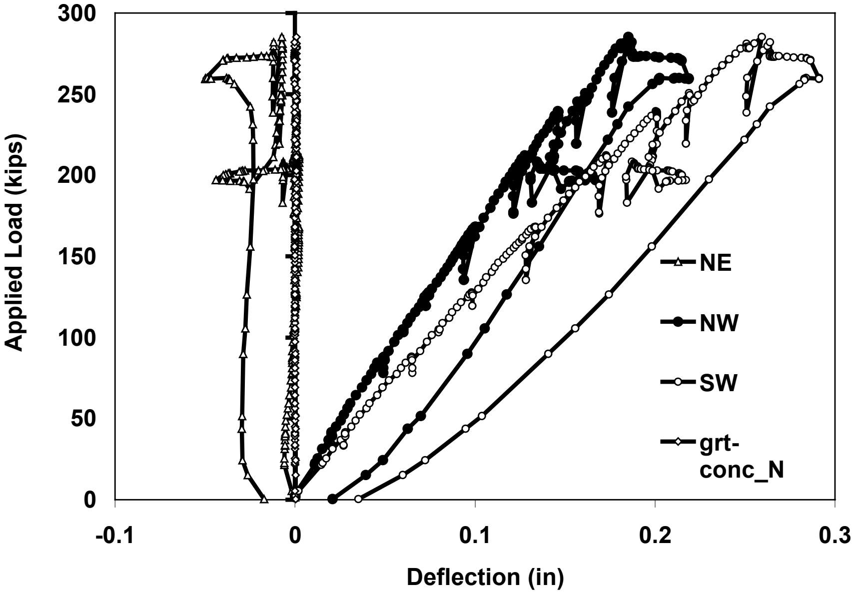
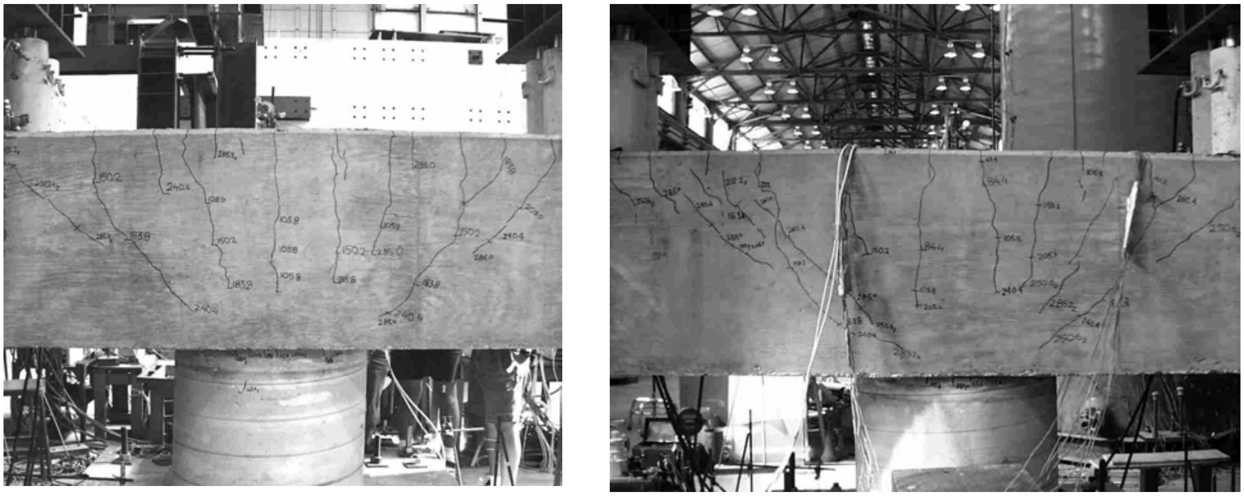


Figure 4.62 Load-Deflection Behavior for CDL2 Cap



a. North Sidewall *b. South Sidewall*

Figure 4.63 CDL2 Crack Patterns at Factored-Level Load

Figure 4.64 shows the load-strain behavior for the connectors. As expected, tensile strains developed in the NE bar, though to a level less than 20% of yield. Cracks that extended through the grout pocket during loading did not appear to impact connection performance.

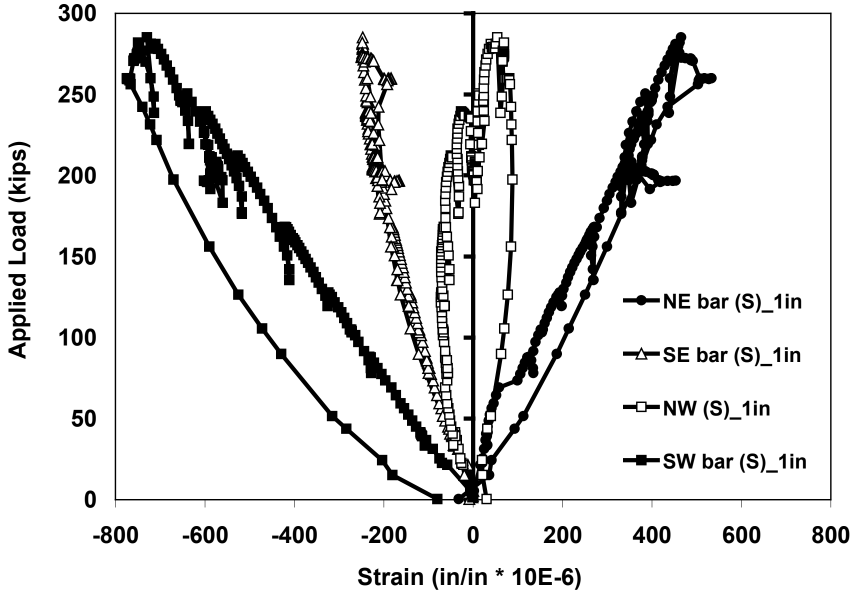


Figure 4.64 Load-Strain Behavior for CDL2 Connectors

The application of a 40-kip horizontal load produced an increase in tensile strain of as much as 120 microstrain in the NE connector. It should be noted that Table 4.6 shows an increase of only 50 microstrain due to a loss in vertical load (from a slow pressure leak) as the horizontal load was applied. The maximum tensile strain accounts for this. Horizontal loading also produced a crack opening at the base of the footing and a horizontal deflection of 0.03 in. Overall effects of the horizontal load were considered relatively minor. No change in pocket cracking was observed.

4.4.5 CDL3

CDL3 used an upper bound eccentricity combination of 7.25 in. transverse and 6.0 in longitudinal. Figures 4.65 and 4.66, respectively, show the larger vertical deflections and strains that developed compared to CDL2. Up to service-level loads, little visible difference in cracking appeared. However, upon application of the 40-kip horizontal load, a crack formed at the base of the column and wrapped around the NE quarter of the perimeter, where tensile stresses due to vertical and horizontal loads combined. As the load was increased to the factored level, another crack appeared on the NE (tension) face of the column near mid-height, as shown in Figure 4.67a. Upon application of the horizontal load, vertical cracks appeared on the west column face (Figure 4.67b), indicating large compressive stresses. In addition, the horizontal column crack and shear cracks in the cap extended. Figure 4.68 shows side and top views of the cap at the factored level. Maximum crack widths at the pocket top were as large as 0.016 in. The increase in crack widths at the surface of the grout pocket exhibited the effect of more severe eccentricities. The maximum tensile strain was again less than 20% of yield, including effects of the horizontal load.

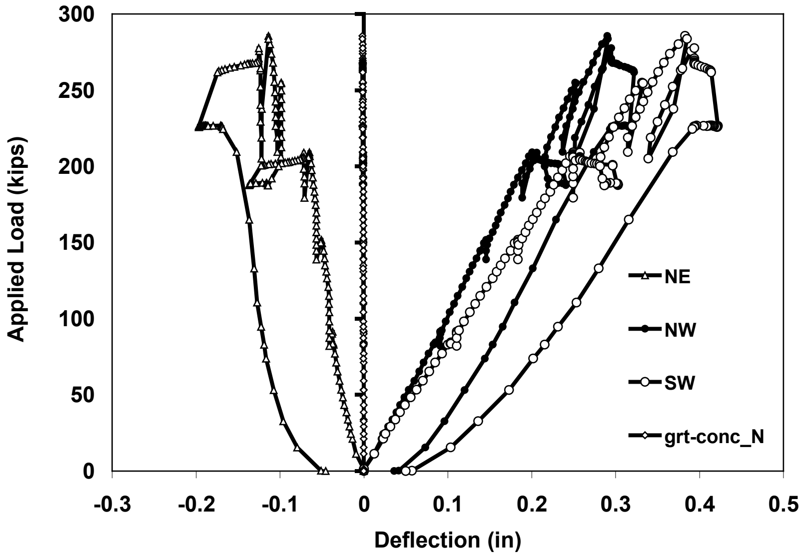


Figure 4.65 Load-Deflection Behavior for CDL3 Cap

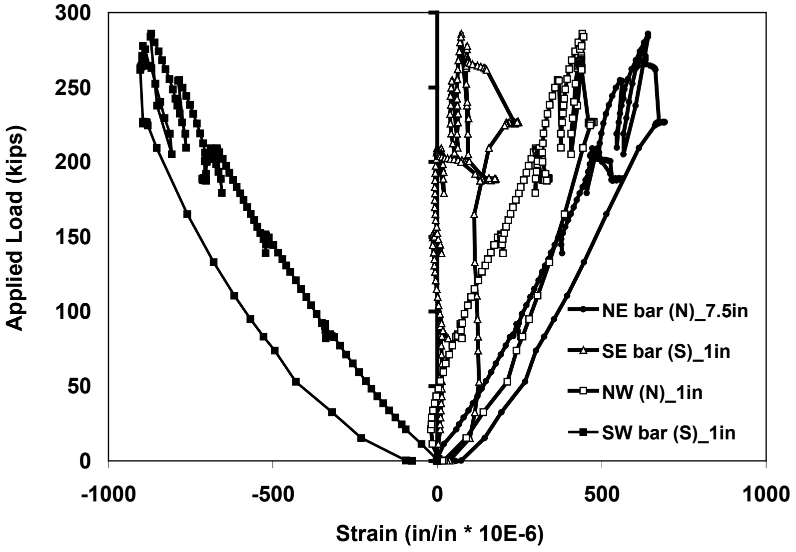
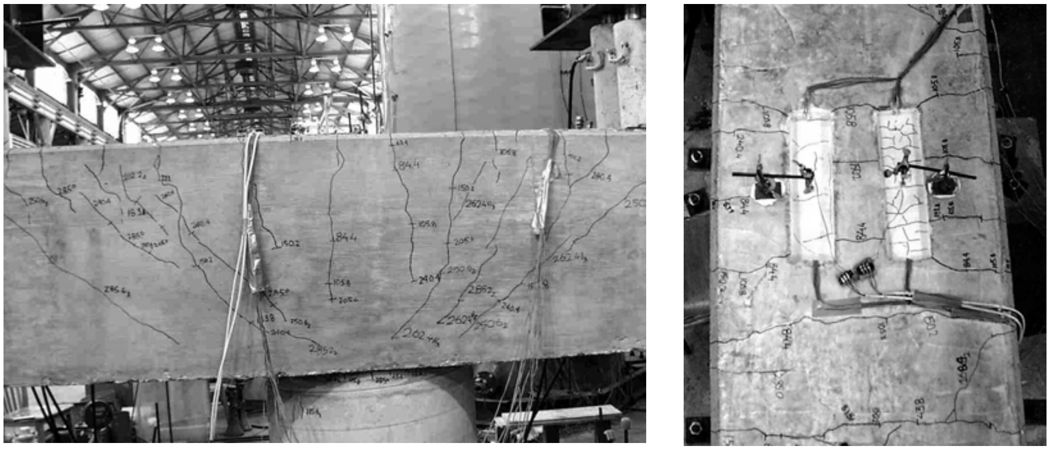


Figure 4.66 Load-Strain Behavior for CDL3 Connectors

a. Crack on Tension Side *b. Cracks on Compression Side*

Figure 4.67 CDL3 Column Crack Patterns at Factored-Level Load



a. South Sidewall *b. Cap Top*

Figure 4.68 CDL3 Crack Patterns at Factored-Level Load

4.4.6 CDL4

CDL4 used single-ram loading on the west side of the cap to apply a large 55.5-in. transverse eccentricity and thereby yield the east side connectors. Figure 4.69 shows that all connectors were in tension, indicating a shift of the neutral axis to the west edge of the column, as expected from Figure 4.60. As the load increased, a gap gradually opened at the east side of the bedding layer. Figure 4.70a shows a ruler penetrating several inches into the bedding layer at a load of 47 kips. Figure 4.70b shows the extension of the preexisting column crack around the east side of the column. In addition, the figure shows a new vertical splitting crack that developed as the SE connector resisted pullout.

At 61 kips, the SE connector yielded at the bedding layer. The NE strain record was unreliable, although yield of the NE bar probably also occurred. Yield occurred at a load approximately 12% less than predicted. For all of the column bent specimens (CDL, CVD, CBC), the capacity was predicted directly from a uniaxial P-M interaction diagram. Some of this difference may be due to the as-built location of the SE connector, which was positioned slightly closer to the west side of the column than assumed in the P-M analysis. It is unlikely that the difference is attributable to a lower bar yield strength than assumed because Figure 4.72d suggests the yield strength was actually larger. The P-M diagram in Figure 4.56

includes the data point for the maximum applied load, which is shown beyond the strength curve. However, Table 4.6 shows a ratio of only 0.88 because the yield load is used to indicate that the bar yielded before it was predicted according to the assumptions in the analysis.

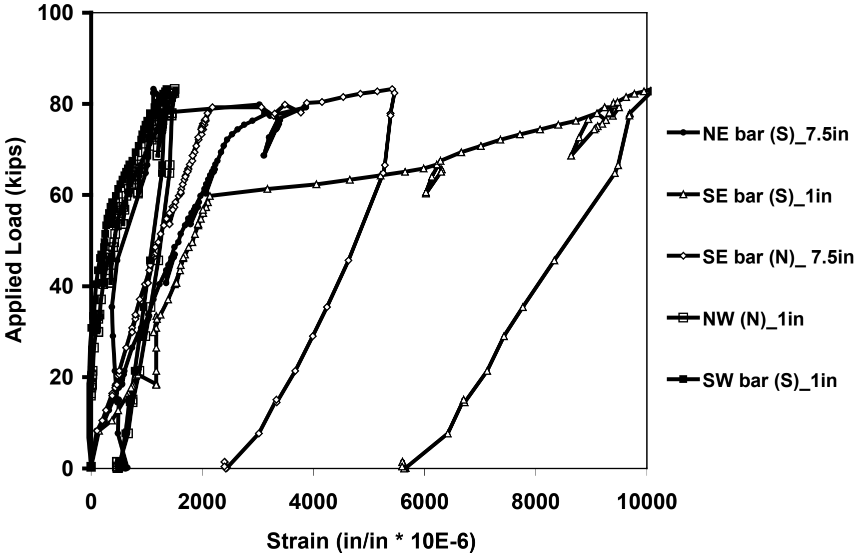


Figure 4.69 Load-Strain Behavior for CDL4 Connectors

a. Bedding and Vicinity *b. Column*

Figure 4.70 CDL4 Crack Patterns on East Side at Factored-Level Load

The load-deflection plot in Figure 4.71 shows the change in stiffness at 61 kips due to bar yield and additional cracks on the east side of the column. The bedding gap grew to a thickness of nearly ¼ in. at 67 kips. After pumping the ram further, the east side connectors at the 1-in. depth began to strain harden. The specimen was then loaded to a maximum of 83 kips to yield the NE and SE connectors at the 7.5-in. depth (Figure 4.69). The specimen tilted down nearly 0.9 in. at the maximum load. Most of the deflection was due to joint rotation at the bedding layer. Column cracks were only 0.005 in. wide.

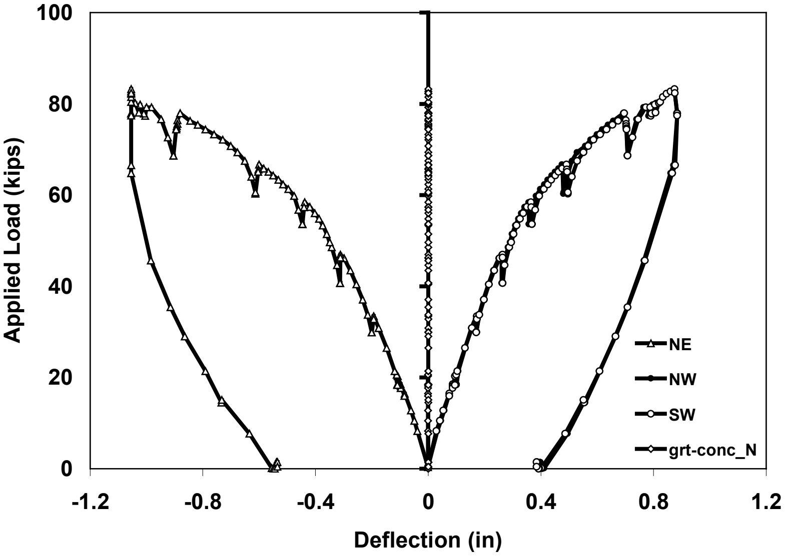
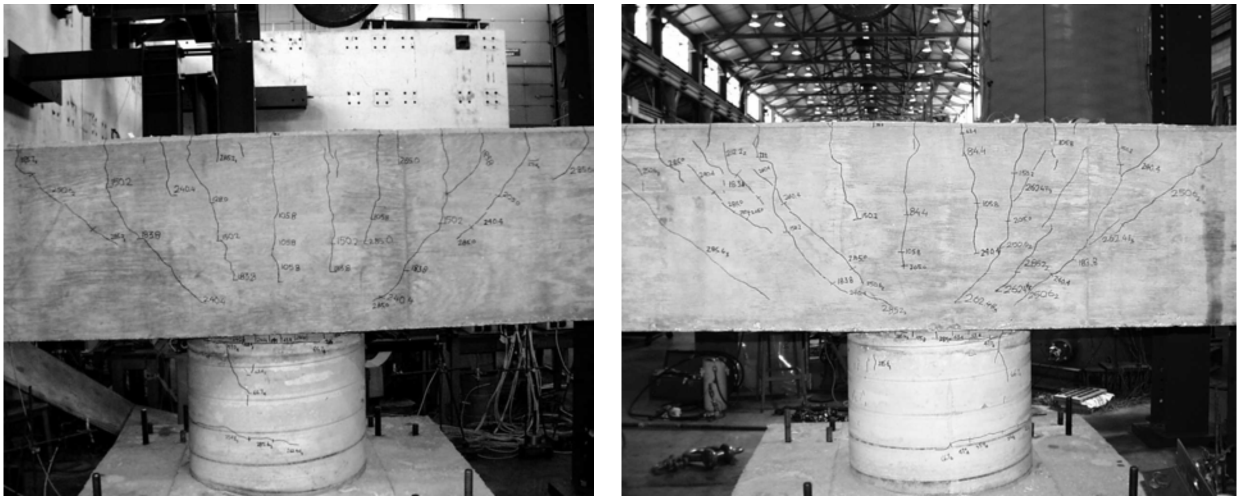


Figure 4.71 Load-Deflection Behavior for CDL4 Cap

An inspection of the connection region at the maximum load revealed the following: 1) crack widths at the top of the grout pockets did not open beyond the 0.016-in. width from CDL3, and 2) no visible distress attributable to bar yield was observed around the connectors, despite flexural and shear cracks in the connection region (Figure 4.72). In addition, east side connectors achieved yield at both 1-in. and 7.5-in. locations without loss of anchorage. However, vertical cracks did form in the column due to splitting stresses associated with connector anchorage.

Figure 4.72c compares the load-deflection response at the west end of CDL4 to a prediction based on a frame analysis using a rigid connection and cracked section properties. The stiffness of the specimen is shown to be smaller by a factor of approximately 2 to 2.4, due primarily to the small number of connectors on the tension side that afford a smaller joint stiffness than the rigid joint assumed in analysis.



a. South End *b. North End*

Figure 4.72 CDL4 Crack Patterns at Factored-Level Load

The moment-curvature plot shown in Figure 4.72d also indicates a difference in stiffness due to the actual number of connectors. Figure 4.72d shows a close match in slopes for the CDL specimen and a prediction based on a sectional analysis at the top of the column, although a softening of CDL response occurred at a low load, perhaps related to the void at the bedding layer (south central area). This limited comparison indicates that response of a precast bent cap system might be reasonably predicted from analysis if the joint stiffness used in an analytical model were based on the actual number and location of connectors. Figure 4.72d also shows the stiffness associated with a cast-in-place column using eight #9 bars around the perimeter. The CDL stiffness is smaller by a factor of approximately 2. The implications of this for design are discussed in the development of a design methodology in Chapter 6.

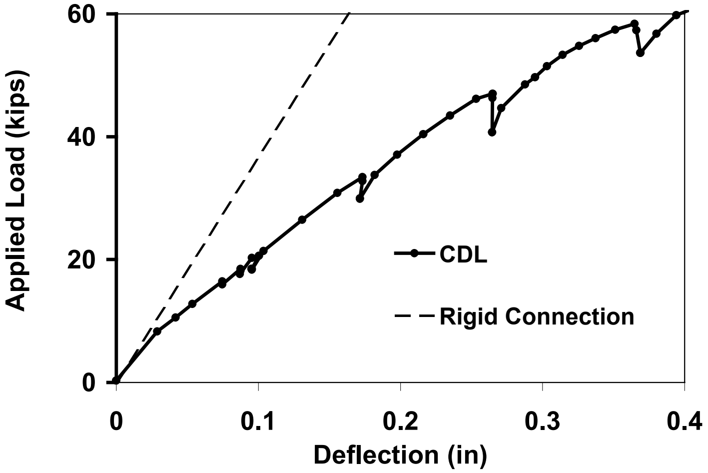


Figure 4.72c Load-Deflection Comparison for CDL and Analytical Prediction

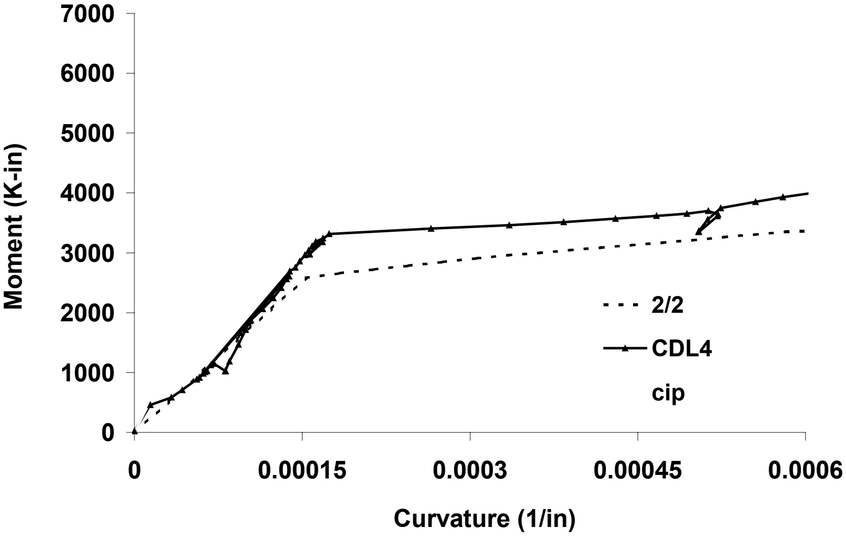


Figure 4.72d Moment-Curvature Comparison for CDL and Analytical Prediction

The connection demonstrated outstanding force transfer characteristics. Excellent anchorage of connectors within the grout pocket was evident as the NE and SE connectors achieved yield at an embedment of $13d_b$. The use of grout with a compressive strength lower than that for the cap concrete (5.1 ksi vs. 6.0 ksi) did not affect anchorage. Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. Excellent ductility was exhibited not only by the achievement of bar yield and strain hardening, but also by the significant spreading of yield along at least 7.5 in. of the connector length. The use of shims and the existence of a bedding layer had no discernable effect on response.

4.5 VERTICAL DUCT AND BOLTED CONNECTION SPECIMENS ON A CAST-IN-PLACE COLUMN

4.5.1 Scope of Testing

Four connection tests were conducted on both a vertical duct specimen (CVD) and a bolted connection specimen (CBC), each using a cast-in-place column. Because the CVD and CBC test series used the same basic load history as CDL, this section compares response for the three specimens, rather than repeating a detailed analysis for each test. Table 4.7 compares the test matrix for all three specimens. The first three tests (Tests 1-3) are considered proof tests of the connection at service and factored levels, while Test 4 was designed to yield connectors on one side of the connection.

4.5.2 Summary of Results

Table 4.7 lists results for CDL, CVD and CBC tests, and Figure 4.73 shows the corresponding P-M strength interaction diagrams with multiple curves to account for the different concrete and connector strengths. As expected from the P_{max}/P_n ratios and Figure 4.73, all specimens exhibited minor levels of connection distress during proof tests. Although some differences were observed between specimens, CVD and CBC response was comparable to that of CDL. Maximum connector tensile strains for CVD and CBC were almost twice as large as for CDL, but maximum strains were still limited to just 30% of bar yield for Test 3. Some of the difference in strain was due to as-built locations for connectors. Bar pullout was not significantly challenged in tests. CDL compressive strains were more than twice that for other specimens. Curvatures, however, did not show significant differences. All specimens showed reasonable agreement with predicted strains. Horizontal loads produced minor increases in strain and deflection.

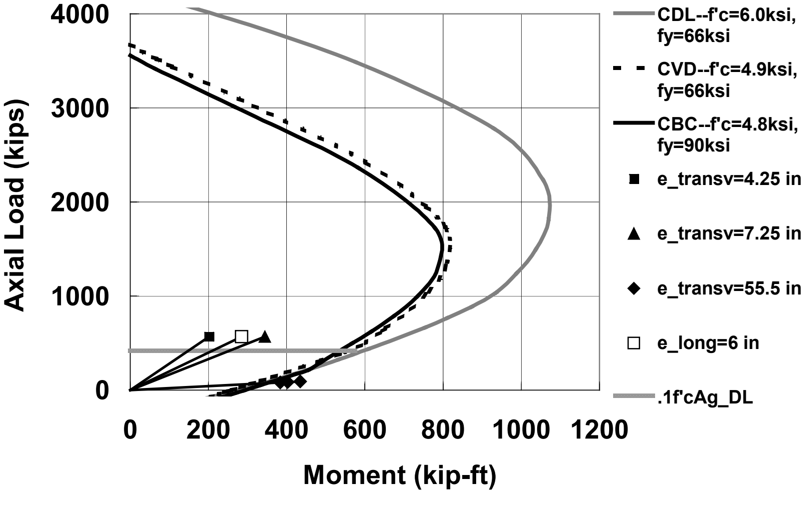


Figure 4.73 Axial Load-Moment Interaction Diagram for CDL, CVD, and CBD

Because of the use of ducts to house connectors for CVD and CBC specimens, flexural cracks were unable to pass through the connectors, in contrast to CDL grout pockets. No cracks appeared at the surface of grouted ducts under any load combination. Crack patterns on the cap top and side faces were very similar for all specimens. Vertical cracks formed at the exposed portion of the bedding layer at service and factored levels, but widths were less than 0.002 in. Slip between the grout, duct, and concrete was negligible in all cases. The presence of shims in the bedding layer had no discernable effect on connection performance for any specimen. Duct strains were relatively minor.

Connectors on the east side yielded at both the 1-in. and 7.5-in. locations in Test 4 for all specimens. Yield loads for CVD and CBC were within 6% of predicted values. Only minor differences in specimen response were observed. CBC strains 1 in. below the cap top were very small, indicating that significant bond developed along the rod and that bolt anchorages at the cap top provide redundancy. Vertical cracks appeared on both the tension and compression faces of columns, due to splitting stresses and compressive stresses, respectively. A large gap gradually opened up on the east side of the bedding layer for each specimen. This was accompanied by large vertical deflections, due primarily to joint rotation. For CBC, the load was increased until crushing occurred at the column top on the compression side.

Load-deflection response for the three specimens was compared to a prediction based on a frame analysis using a rigid connection. The ratio of actual-to-predicted deflection was approximately 2. The CVD specimen exhibited the stiffest response, followed by the CBC and CDL specimens, although behavior was quite similar for all specimens. The difference in stiffness between the specimens and the analytical prediction was attributed to the use of a small number of connectors on the tension side that produced a smaller joint stiffness than the rigid joint assumed in analysis. Analytical and actual moment-curvature response compared closely, indicating that response of a precast bent cap system can be reasonably predicted from analysis if the joint stiffness used in an analytical model accounted for the actual number and location of connectors. Specimen stiffness was approximately half that predicted for a cast-in-place specimen.

The connection demonstrated outstanding force transfer characteristics. Excellent anchorage of connectors within the grout pocket was evident as connectors for all specimens achieved yield at an embedment of $13d_b$. For all specimens, grout with a compressive strength lower than that for the cap concrete was used with no discernable effect. Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. Excellent ductility was exhibited not only by the achievement of bar yield and strain hardening, but by the significant spreading of yield along at least 7.5 in. of the connector length for all specimens.

4.5.3 Test 1

The CVD1 and CBC1 specimens exhibited response similar to the CDL1 specimen. As shown in Figure 4.74, the load-deflection response of the SW end compared favorably for all three specimens, although the CDL1 specimen exhibited a response 15% stiffer. The CDL specimen also developed considerably larger maximum compressive strains. This may be the result of a smaller bearing area of the grout near the SW connector associated with the void at the bedding layer (Section 4.2.4.2). The CBC specimen actually developed tensile strains in the NE connectors (Figure 4.75), although compressive strains were expected from analysis (Figure 4.57). Figure 4.76 compares the predicted and actual (average) connector strains for the east and west bars for the three specimens. Because only transverse eccentricity was used, the slope of the lines represents curvature. Although some scatter is evident, reasonably small differences resulted between strain magnitudes and curvatures. Maximum strain levels were only 15% of yield.

Figures 4.77 and 4.78 show side and top views of the specimens during the CVD1 and CBC1 tests. Side face crack patterns and loads at which cracks developed compare closely to the CDL1 patterns (Figure 4.58). Top views reveal a basic difference between grout pocket connections and connections that use ducts such as CVD and CBC: cracks pass directly through connectors in grout pockets, whereas cracks extend to the ducts but are obviously unable to reach connectors in duct connections. Although cracks in the grout pocket were limited to 0.007 in. at service level, no cracks developed in the CVD and CBC ducts at the factored level. For Test 1 as well as subsequent tests, slip between the grout, duct, and concrete was negligible for all specimens. In addition, the presence of shims at the bedding layer had no discernable effect on connection performance.

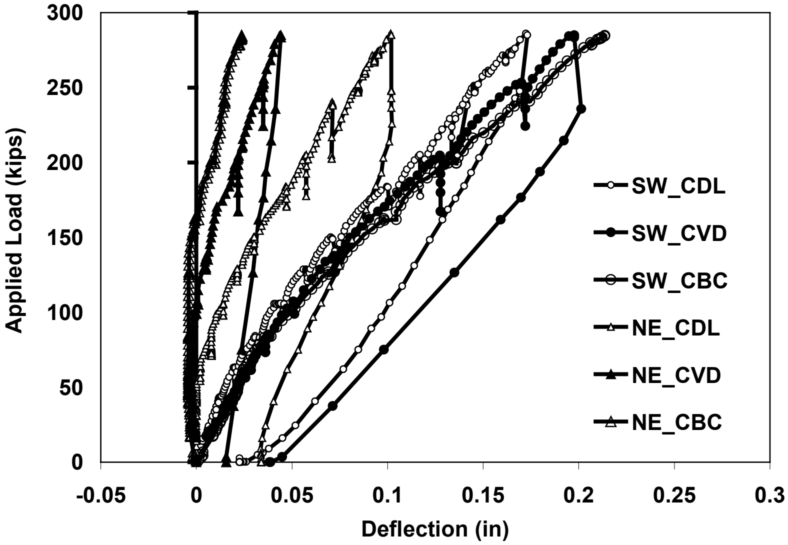


Figure 4.74 Load-Deflection Behavior for Test 1 (CDL, CVD, and CBC)

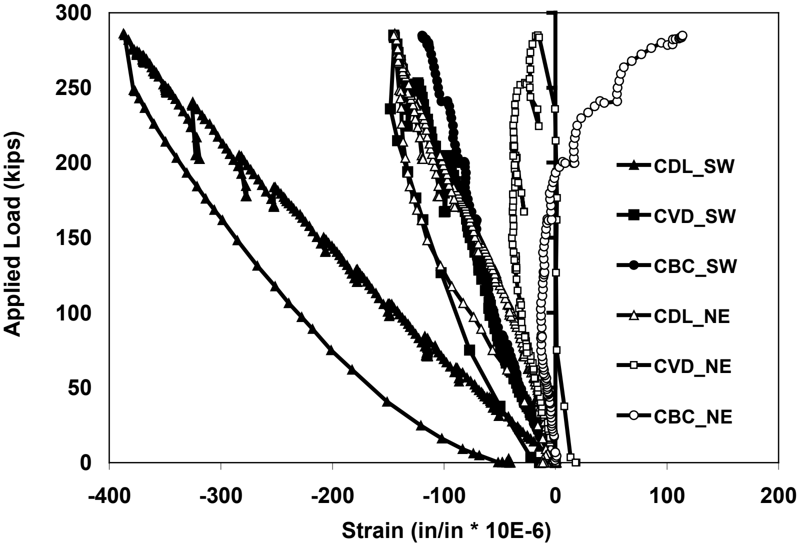


Figure 4.75 Load-Strain Behavior for Test 1 Connectors (CDL, CVD, and CBC)

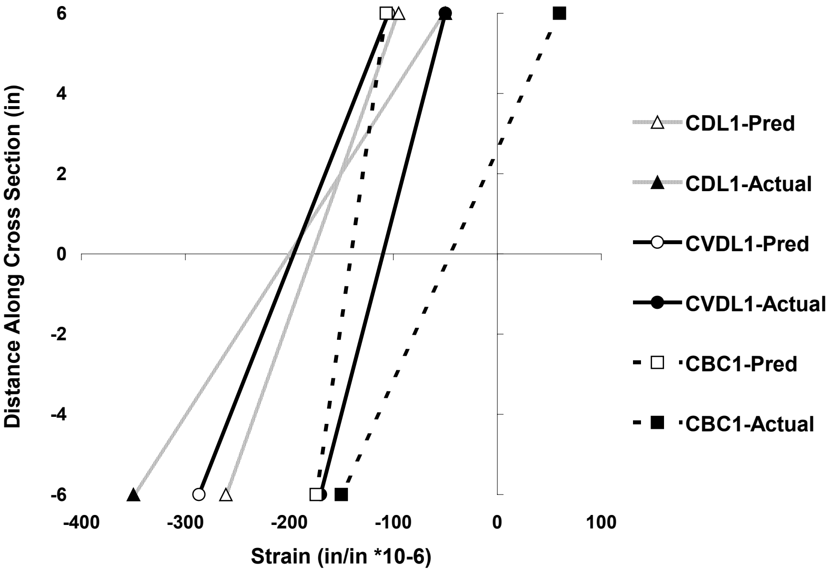
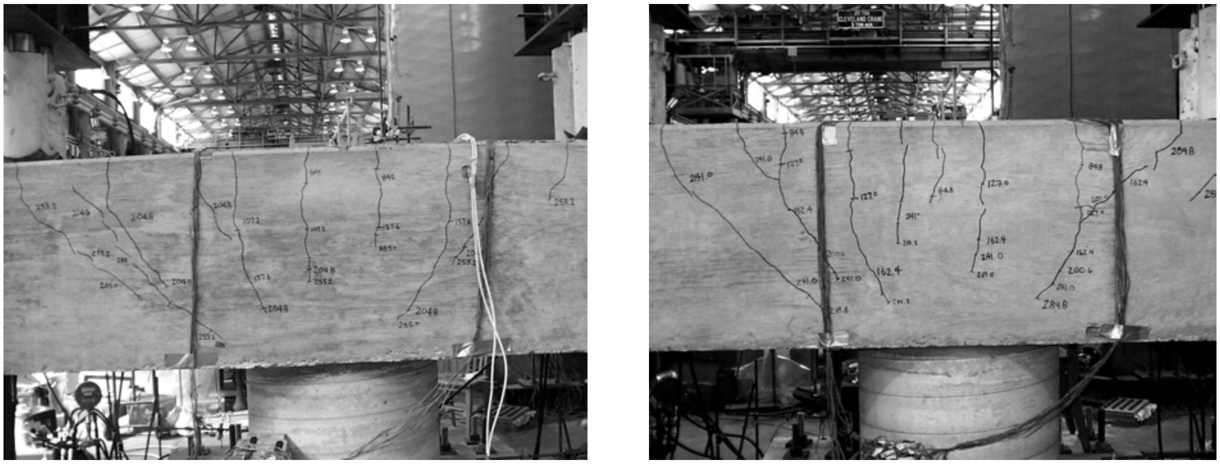


Figure 4.76 Predicted vs. Actual Strain Distribution for Test 1 Connectors at Factored Load

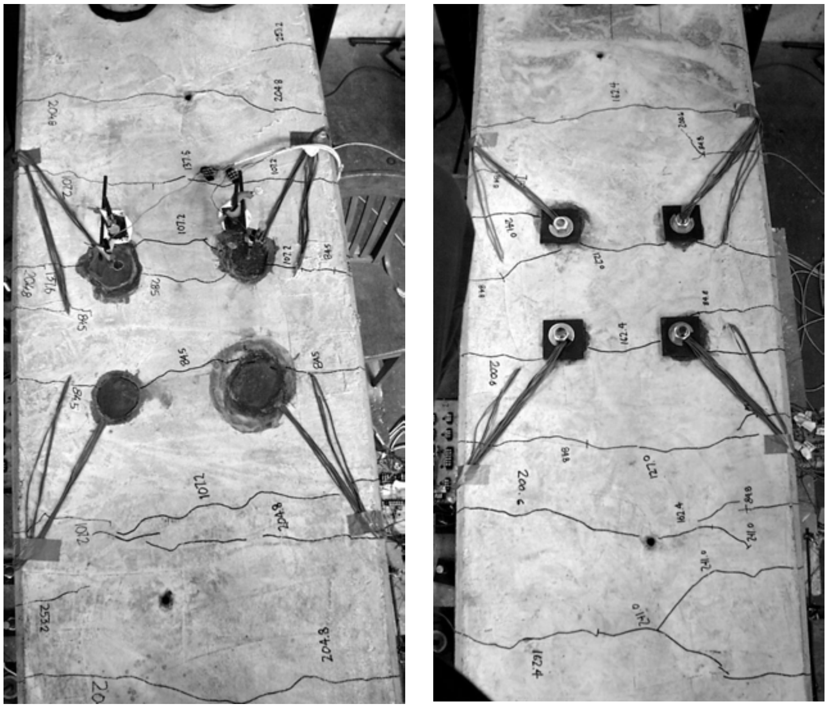


a. CVD *b. CBC*

Figure 4.77 CVD1 and CBC1 Crack Patterns on South Sidewall at Factored-Level Load

4.5.4 Test 2

Using a combined eccentricity of 4.25 in. transverse and 6.0 in longitudinal, the CVD and CBC specimens again exhibited response similar to CDL. Following beam cracking in Test 1, all three specimens exhibited a similar stiffness up to service level in Test 2 (Figure 4.79). For CVD and CBC, a crack at the footing-column interface developed as the load was increased to service level. This may have contributed to the slight differences in response that occurred upon application of the horizontal load. Figure 4.79 also shows a momentary loss of load in CVD during application of the horizontal load at service level. This corresponded to a slight tipping of the vertical rams.



a. CVD *b. CBC*

Figure 4.78 CVD1 and CBC1 Crack Patterns on Cap Top at Factored-Level Load

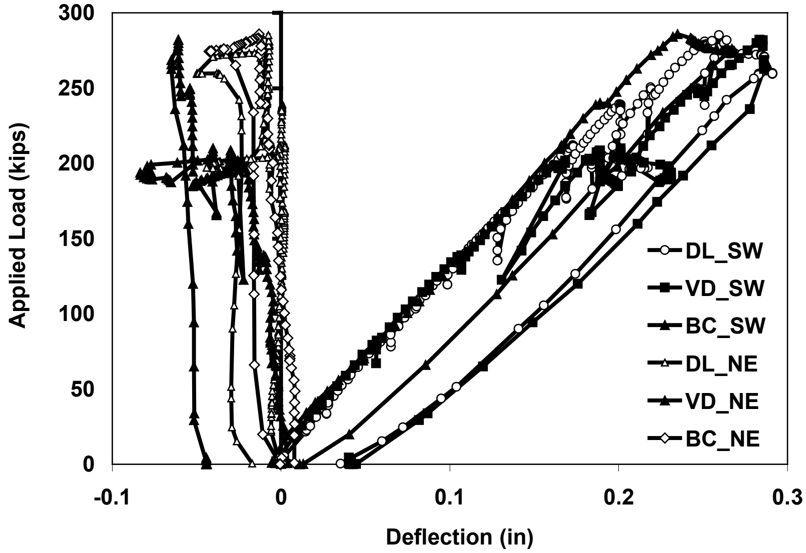


Figure 4.79 Load-Deflection Behavior for Test 2 (CDL, CVD, and CBC)

As shown in Figure 4.80, CDL developed significantly larger compressive strains, but smaller tensile strains. Maximum tensile strains were comparable for all cases, and were less than 25% of yield. Delta strains were no more than 250 microstrain. Crack patterns changed very little from Test 1 and are therefore not shown.

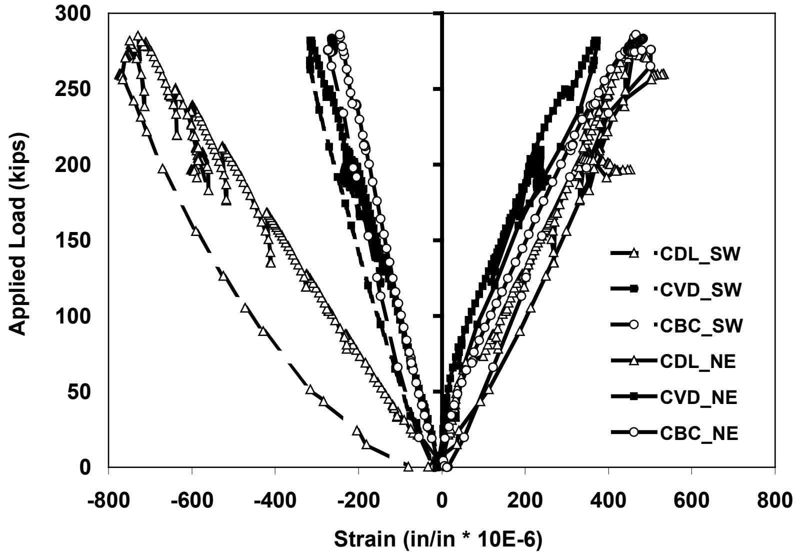


Figure 4.80 Load-Strain Behavior for Test 2 Connectors (CDL, CVD, and CBC)

4.5.5 Test 3

Test 3 used a combined eccentricity of 7.25 in. transverse and 6.0 in. longitudinal. This combination is considered the maximum realistic combination (i.e., an upper bound) for bents using cast-in-place columns. Figure 4.81 exhibits little difference in load-deflection response for the three specimens and the same relative stiffness between specimens discussed for Test 2. CDL showed a slightly softer response. Figure 4.82 shows a minor extension of flexural cracks and formation of several new shear cracks on the side face for CVD and CBC. Figure 4.83 reveals little difference in crack pattern at the cap top compared to Test 1. CDL also showed little difference compared to CDL. However, crack widths at the surface of the grout pocket did noticeably increase, which contributed to larger frame deflections.

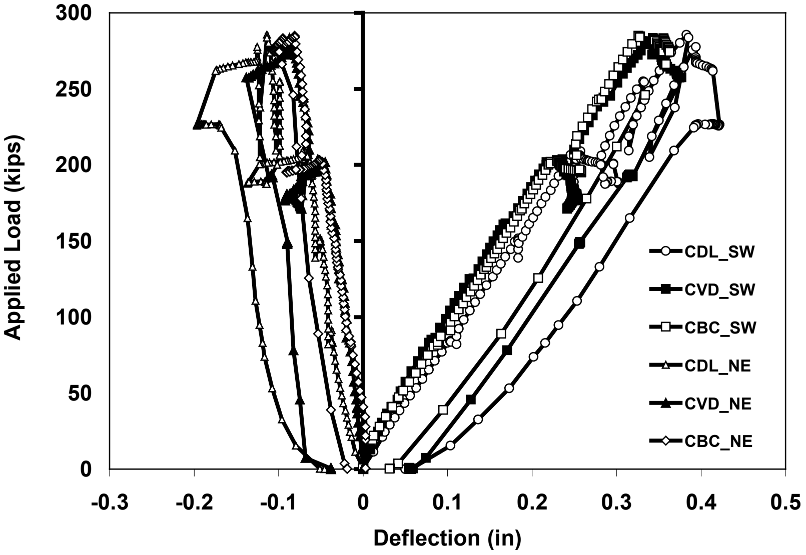
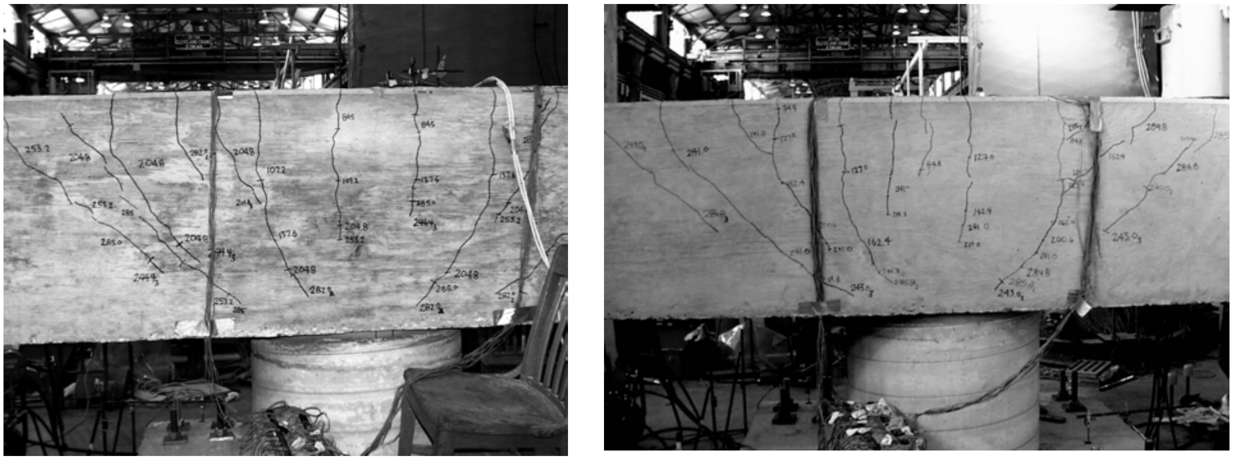
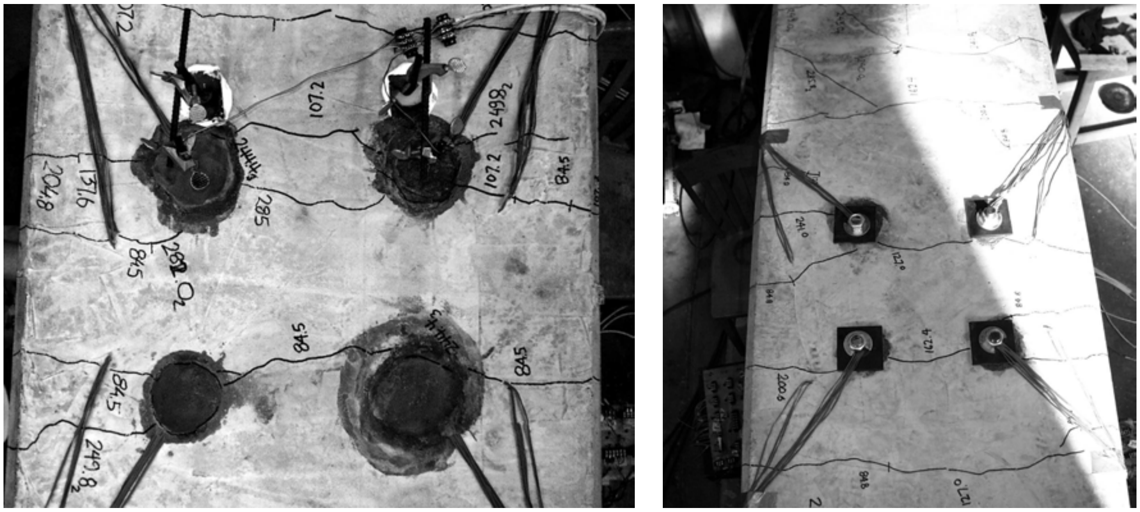


Figure 4.81 Load-Deflection Behavior for Test 3 (CDL, CVD, and CBC)



a. CVD *b. CBC*

Figure 4.82 CVD3 and CBC3 Crack Patterns on South Sidewall at Factored-Level Load



a. CVD *b. CBC*

Figure 4.83 CVD3 and CBC3 Crack Patterns on Cap Top at Factored-Level Load

As shown in Figure 4.84, CDL again developed larger compressive strains, 40% of yield and more than twice those of CVD and CBC. However, CVD and CBC achieved tensile strains more than twice as large as CDL. Maximum tensile strains were only 30% of yield. CBC strains near the top of the anchorage (1 in. from the cap top) were negligible. Delta strains were the maximum that developed for any proof test, but were no more than 310 microstrain.

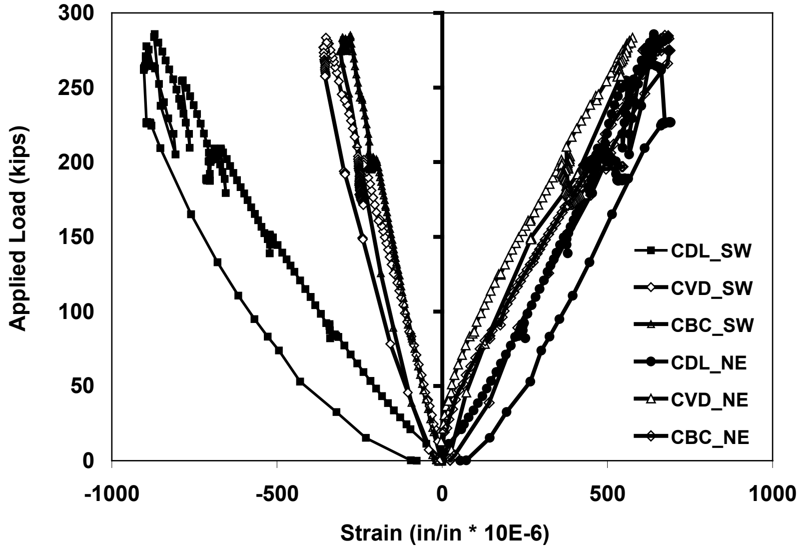


Figure 4.84 Load-Strain Behavior for Test 3 Connectors (CDL, CVD, and CBC)

4.5.6 Test 4

Using a 55.5-in. transverse eccentricity, Test 4 produced yield in the east connectors for all specimens. Figure 4.73 shows that bar yield was expected at a load under 100 kips. Table 4.7 shows that yield loads for CVD and CBC were within 6% of predicted values. Large vertical deflections were produced at yield, as shown in Figure 4.85. CVD exhibited stiffer response at all load levels. Significantly softer CDL response at loads greater than 60 kips is attributed to bar yield.

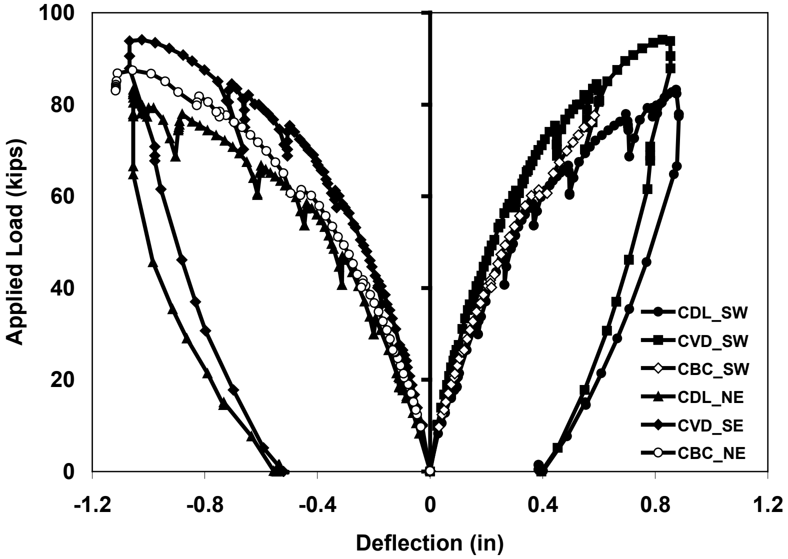


Figure 4.85 Load-Deflection Behavior for Test 4 (CDL, CVD, and CBC)

All specimens were loaded until connectors at both the 1-in. and 7.5-in. locations yielded. As shown in Figure 4.86, bar strains at the 1-in. location reached over 10,000 microstrain (1%) before the specimens were unloaded. The CVD strain gages at 7.5 in. broke prior to yield. CBC exhibited a softer load-strain response, and yield of the bolts was not defined by a flat plateau. CBC strains 1 in. below the cap top (not shown) were only 200 microstrain. This indicates that significant bond developed along the rod for anchorage. Thus, bolt anchorages at the cap top provide redundancy. Figure 4.87 compares the predicted and actual connector strains for the three specimens at the load corresponding to first yield of a connector (61 kips for CDL). Actual strains were smaller than predicted, and reasonably small differences resulted for all cases.

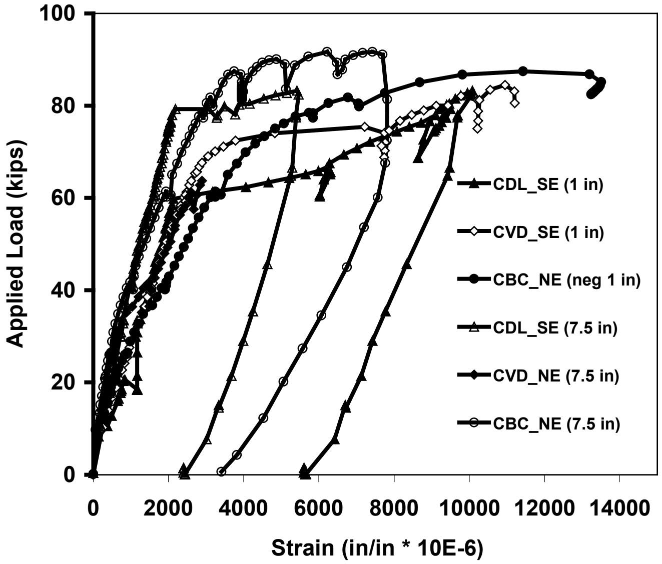


Figure 4.86 Load-Strain Behavior for Test 4 Connectors (CDL, CVD, and CBC)

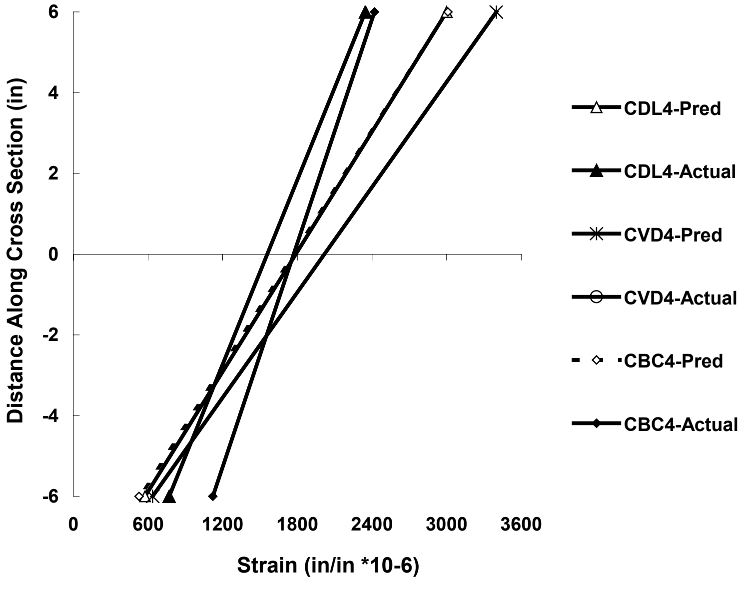
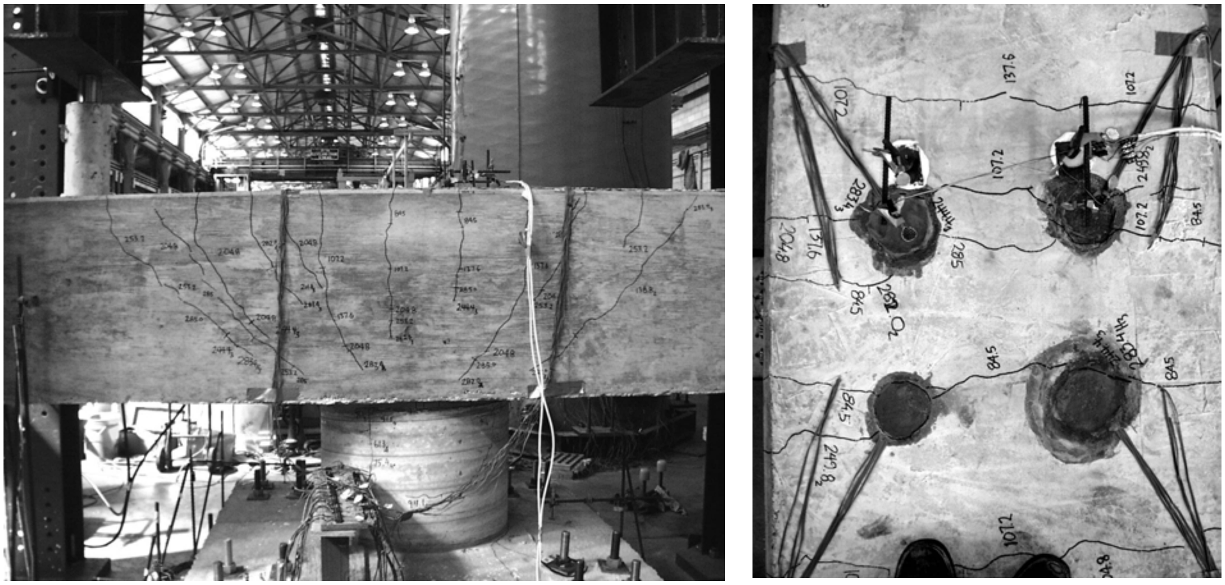


Figure 4.87 Predicted vs. Actual Strain Distribution for Test 4 Connectors at 61 Kips

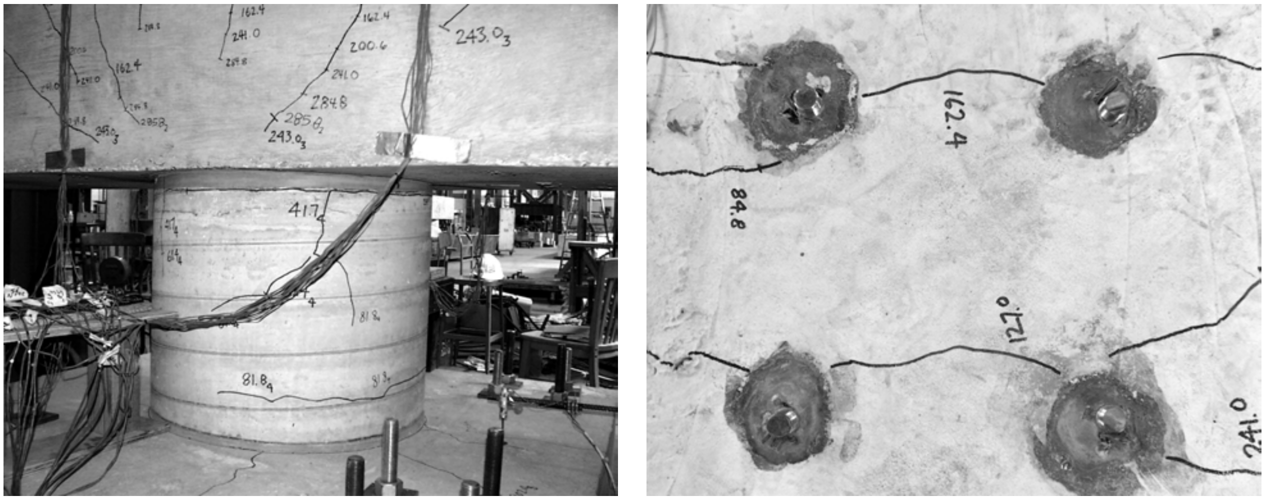
Figure 4.88 shows the CVD4 crack patterns on the side face and top at the maximum load. Figures 4.88a and 4.89a show vertical cracks that developed on the compression and tension sides of the column, in addition to horizontal flexural cracks. Vertical cracks on the tension side were produced by splitting stresses that developed as the connector resisted pullout, whereas vertical cracks on the compression side and in the bedding layer were caused by a combination of large compressive stresses and lateral restraint.

CDL and CBC exhibited similar cracks. As expected, grout in the CVD and CBC ducts did not crack under any of the load cases (Figures 4.88b and 4.89b).



a. South Sidewall *b. Cap Top*

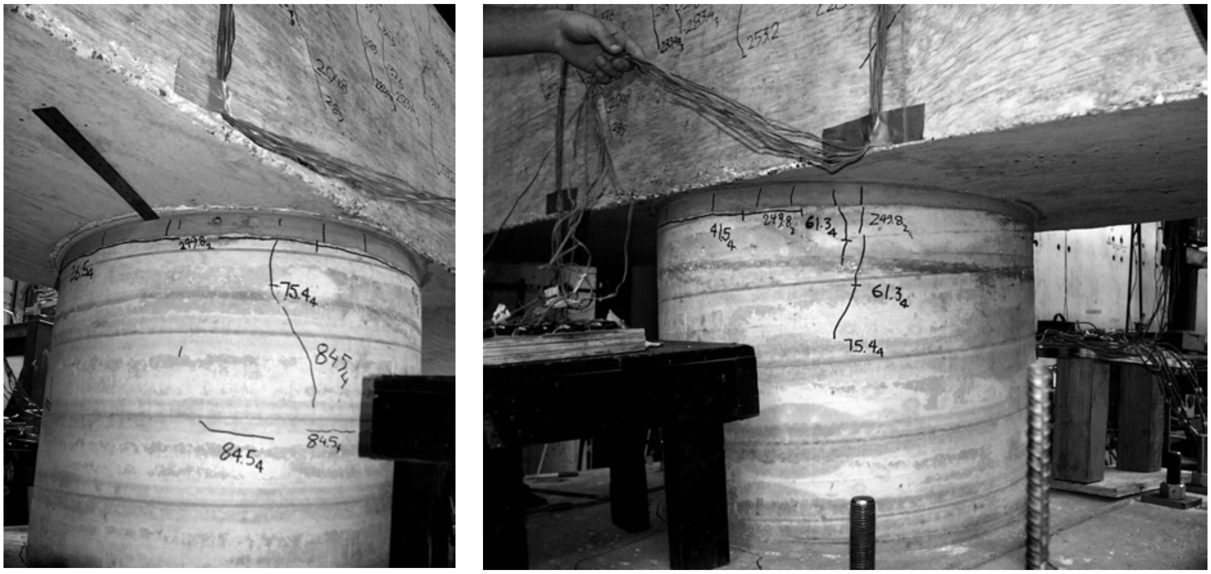
Figure 4.88 CVD4 Crack Patterns at Factored-Level Load



a. South Sidewall *b. Cap Top*

Figure 4.89 CBC4 Crack Patterns at Factored-Level Load

Figures 4.90 and 4.91 show the tension and compression sides of the bedding layer for CVD and CBC, respectively. As observed for CDL, a gap gradually opened up at the east side of the bedding layer. Figures 4.90a and 4.91a show this gap near the maximum load. For CDL and CVD, the specimen was unloaded after bar yield was achieved at the 7.5-in. depth. However, for CBC, load was increased further until crushing occurred on the compression side of the column. Figure 4.91b shows the spalled concrete and grout at the column top after crushing occurred. The cap tilted more than 0.8 in. and the bedding gap was ~1/4 in. for all specimens at the failure load.



a. Cracks on Tension Side *b. Cracks on Compression Side*

Figure 4.90 CVD4 Bedding and Column Crack Patterns at Factored-Level Load

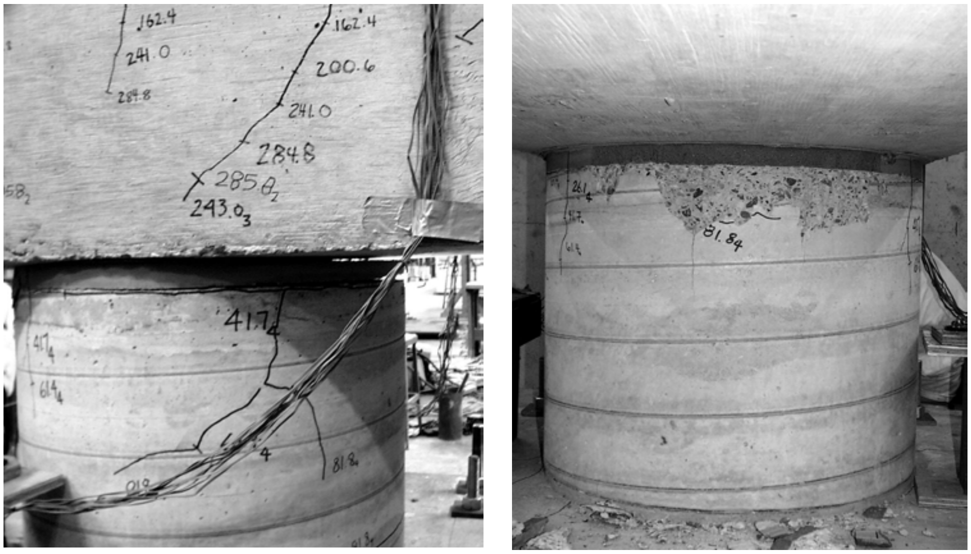


Figure 4.91 CBC4 Bedding and Column Crack Patterns at Factored-Level Load

Figure 4.92 compares the load-deflection response for the three specimens to a prediction based on a frame analysis using a rigid connection and an estimate of cracked section properties taken as the same for all specimens. Deflection up to bar yield is shown. Each specimen exhibited a slightly nonlinear load-deflection behavior that was quite similar to the other specimens. The CVD specimen exhibited the stiffest response, followed by the CBC and CDL specimens. The ratio of actual-to-predicted deflection varied between approximately 1.8 and 2.4. Some variation is expected due to as-built locations of connectors as well as steel and concrete material properties. In addition, the CBC specimen used bars that were higher strength but smaller than the other specimens. The difference in stiffness between the specimens and the analytical prediction is due primarily to use of a small number of connectors on the tension side that produces a smaller joint stiffness than the rigid joint assumed in analysis.

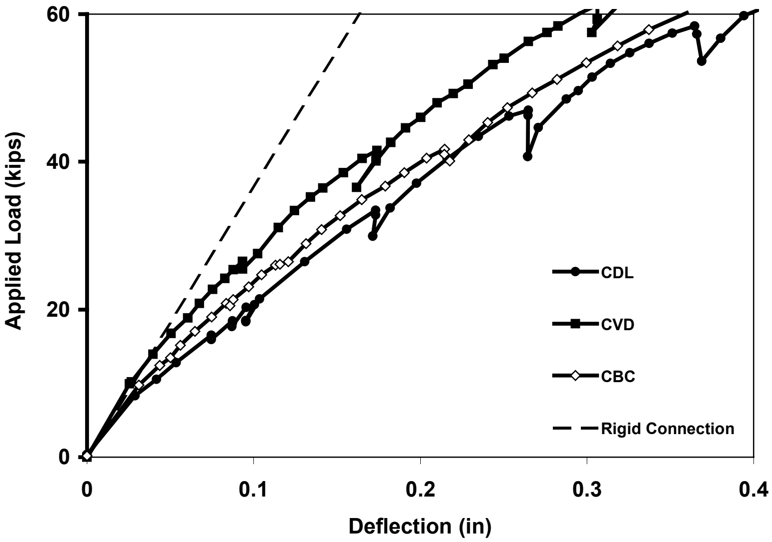


Figure 4.92 Load-Deflection Comparison for CDL and Analytical Prediction

As shown in Figure 4.93, analytical and actual moment-curvature response compared reasonably well. The CBC specimen initially exhibited a very stiff response, due to the tightening of the bolts. This post-tensioning effect was lost upon further loading. The slope of the analytical and actual response matches very closely near the assumed bolt yield stress. The CVD specimen exhibited a stiffer very similar to the analytical prediction, although a slightly larger initial stiffness was observed, as well as a larger yield stress. CDL response compared closely to CVD specimen response. This comparison indicates that response of a precast bent cap system can be reasonably predicted from analysis if the joint stiffness used in an analytical model accounted for the actual number and location of connectors. Specimen stiffness was approximately half that predicted for a cast-in-place specimen. The implications of this for design are discussed in the development of a design methodology in Chapter 6.

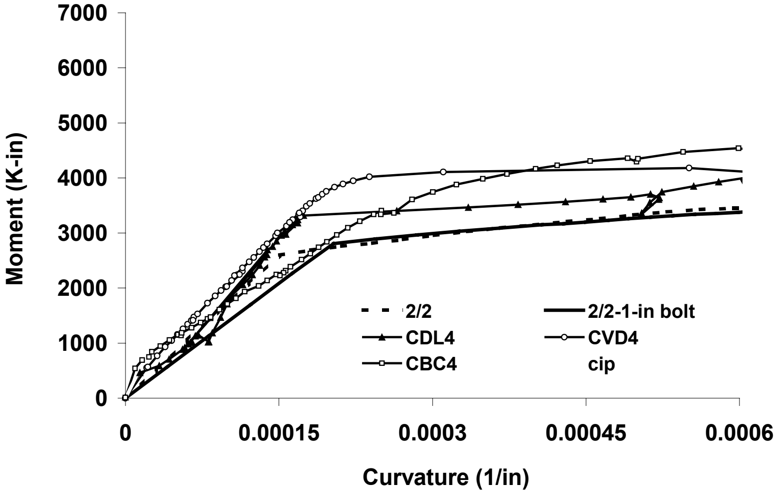


Figure 4.93 Moment-Curvature Comparison and Analytical Prediction

The connection demonstrated outstanding force transfer characteristics. Excellent anchorage of connectors within the grout pocket was evident as connectors for all specimens achieved yield at an embedment of $13d_b$. For all specimens, grout with a compressive strength lower than that for the cap concrete was used with no discernable effect. Minor slip of connectors within the grout pocket is inferred based on the lack of cracking or other distress in the cap, as well as the stiffness exhibited by the load-deflection response of the specimen. Excellent ductility was exhibited not only by the achievement of bar yield and strain hardening, but by the significant spreading of yield along at least 7.5 in. of the connector length for all specimens. The use of shims and the existence of a bedding layer had no discernable effect on response.

Duct strains were relatively small for proof tests, but considerably larger when bar yield was reached. Figure 4.94 compares CVD bar and duct strains on the tension side for 1-in. and 7.5-in. locations. Bar strains are shown to increase at a much greater rate than duct strains. Figure 4.95 shows that CBC duct strains increased much more than in CVD at the 1-in. location, but less at the 7.5-in. location. No clear pattern developed. However, duct strains never reached yield and ducts exhibited acceptable performance for all tests.

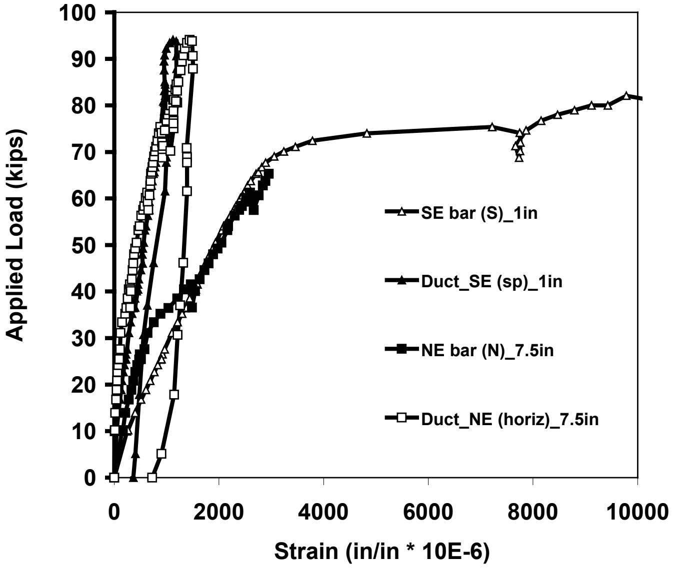


Figure 4.94 Load-Strain Behavior for CVD Bars and Ducts

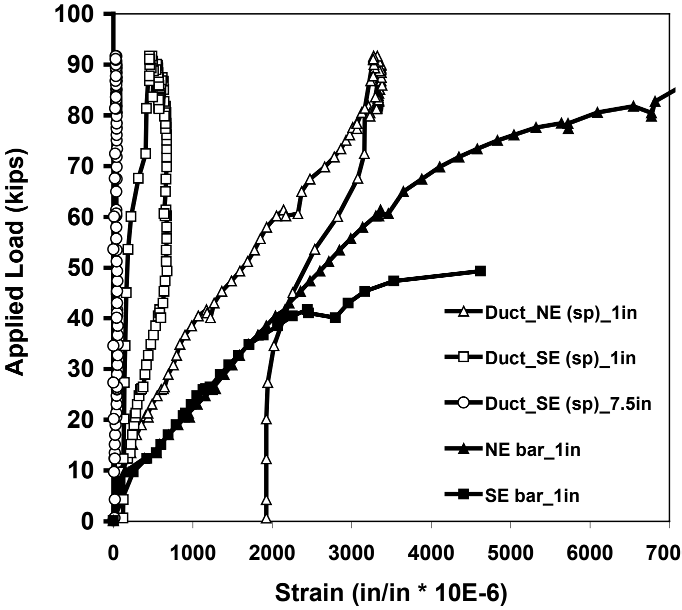


Figure 4.95 Load-Strain Behavior for CBC Bars and Ducts

4.6 CONCLUSIONS

Based on full-scale, connection tests for a single-line grout pocket on a pile, and double-line grout pocket, grouted vertical duct, and bolted connections on a cast-in-place column, the following conclusions can be made:

1. Connection types—*Single-line grout pockets, double-line grout pockets, grouted vertical ducts, and bolted connections are all acceptable alternatives for a precast bent cap system.* Using a minimum number of straight, epoxy-coated #9 bars at a $13d_b$ embedment depth or 1-in. bolts anchored at the cap top, all connection types exhibited the expected strength, as well as excellent ductility and connector anchorage. Connection distress under factored proof loads was minor. In addition, the construction sequence for each test specimen was accomplished with relative ease. With proper grouting of connections, unusual durability concerns are not expected. Section 4.2.4.4 summarizes lessons learned in cap setting and grouting operations.

2. General behavior for proof tests—*Minor levels of connection distress are expected for all connection types in a precast bent cap system.* Proof tests of all specimens produced minor connection distress for factored vertical loads at eccentricities as large as 7.25 in. in the transverse direction and 6 in. in the longitudinal direction (column bent), together with a small lateral load. Bar pullout of connectors was not significantly challenged because the maximum tensile strain in connectors was limited to 30% of bar yield for all specimens. Flexural cracks as large as 0.016 in. formed within grout pocket connections at the factored level. Crack widths were limited to 0.007 in. for initial service-level loading. Flexural cracks were arrested by ducts for grouted vertical duct and bolted connections; cracks did not appear at the grout surface of ducts. The formation of flexural and shear cracks in the connection region did not adversely affect anchorage or the ability of the connection to transfer forces. A surface sealant can be used to minimize durability concerns related to exposed grout surfaces. Under realistic eccentricity and load combinations, failure of the cap or column is expected to precede connection failure.

3. Behavior for failure tests—*Anchorage sufficient to develop 1.25 times the bar yield strength is expected at an embedment depth of* $13d_b$ *for epoxy-coated bars (concrete compressive strength of 6.0 ksi; modified grout cube strength as small as 4.0 ksi). In addition, adequate interlock between connectors, grout, ducts, and the concrete cap is expected for all connection types. Excellent ductility is anticipated.* Application of an unrealistically large eccentricity was required to produce bar yield in connectors. Failure tests for most specimens produced bar yield initially at the bedding layer. Yielding of the connectors then spread over a length of at least 7.5 in. into the connection region of the bent cap as well the columns or piles. This indicates excellent ductility. Based on an embedment depth of $13d_b$, anchorage was maintained without evidence of distress (e.g., splitting cracks) in the region where the connectors were anchored in grout pockets or grouted ducts. In addition, slip between connectors, grout, ducts, and the concrete cap was minor for all connection types (based on the stiffness exhibited by the load-deflection response of the specimen, as well as a the lack of distress in the region of anchorage). Crushing at the compression face of the column was produced following bar yield for one specimen. Large cap deflections and section curvature occurred at the maximum load due to yield of the connectors at the cap-to-column interface.

4. Embedment depth and connector anchorage—*Typical bent cap sizes possess sufficient depth for adequate anchorage of epoxy-coated, straight bars. Embedment depths for epoxy-coated, straight bars should be at least* $13d_b$. *Bond along the threaded rod provides adequate anchorage; bearing at cap top provides redundancy. Connectors should be adequately anchored into both the bent cap and pile or column.* Although proof tests demonstrated that bar yield is not likely for connectors, connectors should conservatively be anchored to develop 1.25 times the yield strength. Failure tests demonstrated that an embedment depth of $13d_b$ was adequate to anchor epoxy-coated, straight bars. Strain records near the cap top for threaded rods indicated that

adequate bond developed along the rod for anchorage; thus, anchorages for bolt bearing at the cap top will provide redundancy. Formation of horizontal flexural cracks as large as 0.01 in. near the pile top and vertical splitting cracks adjacent to connectors in the columns indicated the importance of providing adequate anchorage for connectors in piles and columns.

5. Confining reinforcement—*Spiral confining reinforcement is not expected to contribute to ductility and strength for large connector and duct spacing. However, a minimum amount of spiral reinforcement in the connection region of the bent cap is conservatively recommended for use. Confining reinforcement can also be used to limit effects of connector pullout in piles and columns.* Proof tests demonstrated that small tensile forces developed in connectors, thus limiting the potential ductility and strength contributions of confining reinforcement. Even for bar yield in failure tests, splitting cracks due to pullout were not observed at any surface of the cap, rendering confining reinforcement ineffective. Use of confining reinforcement is considered a conservative provision when connector and duct clear spacing is equal to or greater than that used for tests (2 to 2.5 bar diameters for connectors; 2 to 3 duct diameters for ducts). (See Section 6.3.7.2.) For closer connector or duct spacing, confinement reinforcement may be necessary. Spiral reinforcement requires minimal fabrication and construction effort. Chapter 6 provides a further discussion of this topic. In failure tests, vertical splitting cracks formed on the tension face of columns adjacent to connectors. Thus, column and pile reinforcement should be checked to ensure adequate resistance to splitting is provided.

6. Shims—*Steel or plastic shims are a viable means to support a precast bent cap on piles or columns and are not expected to affect connection response.* The use of steel and plastic shims did not produce any discernable effect on connection response in proof tests or failure tests. Construction issues are discussed in Section 4.2.4.4.

7. Bedding layer—*The existence of a bedding layer is not expected to affect connection response. Possible cracks at the exposed surface of the bedding layer are not expected to pose structural or durability concerns. Connections that use a pile or column embedded into the cap are expected to exhibit response similar to connections using a surface-flush pile or column.* Vertical cracks formed at the exposed surface of the bedding layer at service and factored levels for proof tests, but crack widths were limited to 0.002 in. and thus are not of concern. No cracks were observed on the bottom grout surface at the cap-to-pile interface for the embedded pile specimen during proof tests. In addition, no major difference in response occurred for embedded or surface-flush connection options. A further comparison is made in Phase 3.

8. Grout type and strength—*Masterflow 928, Sika 212, and Euclid Hi-Flow neat grouts provided acceptable force transfer characteristics for application to precast bent cap connections. The use of grout with a lower strength than the surrounding concrete did not adversely affect connection performance. However, it is recommended that the grout strength exceed the concrete strength by a minimum margin, as discussed in Chapter 7.* Despite the low strength of EHF grout (4.0 ksi on test day) and segregation of Sika 212 grout, adequate transfer of forces occurred between connectors, grout, ducts, and concrete for all connection types, as evidenced by development of connector yield strength at an embedment depth of $13d_b$, as well as minor slip and connection distress. A void at the bedding layer of the double-line grout pocket connection may have contributed to an increase in compressive strain at the SW connector and soft initial moment-curvature response. Although modified grout cube strengths were lower than cap and/or column concrete strengths for most tests, this did not adversely affect connection performance. It is conservatively recommended that modified grout cube strength exceed the concrete compressive strength by a 1000-psi margin (see Chapter 7).

9. Analytical predictions—*Traditional approaches such as the Reciprocal Load Method can be used to determine connection strength for combined flexure and axial force effects, including the*

effects of biaxial bending. Analytical models can be used to predict moment-curvature response for a particular connector arrangement. Prediction of reasonable load-deflection response depends on an accurate determination of connection stiffness, which should be based on actual connector configurations. Failure loads were predicted within 15% for all specimens using axial load-moment (P-M) strength interaction diagrams. The Reciprocal Load Method (RLM) was used to predict the load that would produce connector yield for the pile bent specimen (PSL) under biaxial bending. For the column bent specimens (CDL, CVD, CBC), load was applied using only a transverse eccentricity (i.e., uniaxial bending) to produce connector yield. Thus, for these specimens the capacity was predicted directly from a uniaxial P-M interaction diagram. The predicted moment-curvature response matched actual response reasonably well for all specimens. Because excellent connector anchorage was achieved for all connections and the bedding layer did not appear to affect response, the number and location of connectors at the bedding layer is expected to be the main factor affecting connection rigidity. Moment-curvature response for different connector configurations can be used to help define connection rigidity for frame analysis of a precast bent cap system. This topic is addressed further in Chapter 6.

4.7 REFERENCES

4.1. Sumen, G., "Testing of Precast Bridge Bent Cap Connection Details," MS Thesis, The University of Texas at Austin, Austin, TX, August 1999.

4.2. US 287 Northbound over IH-30, Plan Sheets, Texas Department of Transportation, Bridge Design Division, February, 1993.

4.3. Telephone discussion with Lloyd Wolf, Texas Department of Transportation, Bridge Design Division, Austin, Texas, September, 1997.

4.4. Matsumoto, E.E.; Waggoner, M.C.; and Kreger, M.E., "Development of Precast Bent Cap Systems and Testing Program," *Interim Report 1748-1*, Center for Transportation Research, The University of Texas at Austin, March 1998.

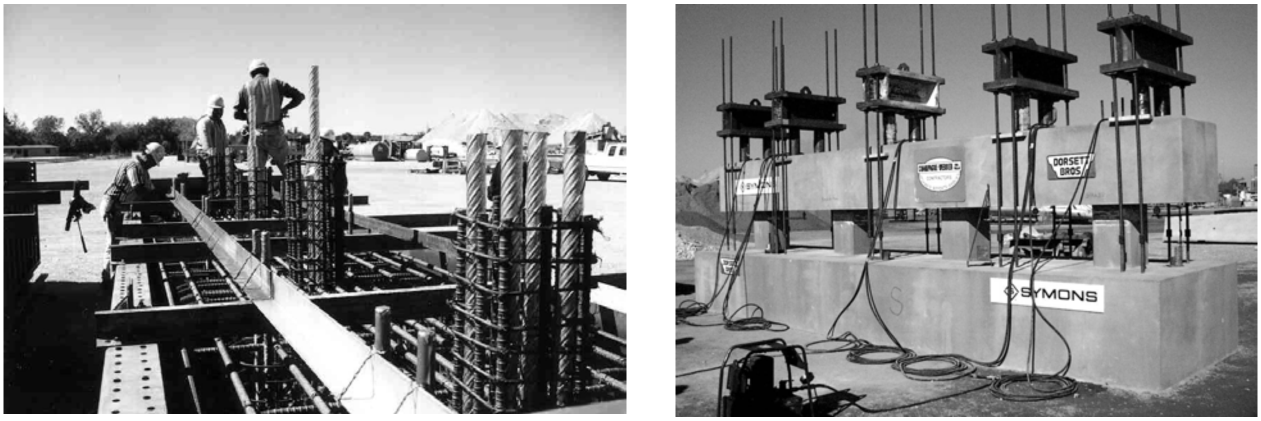
4.5. Nilson, A., *Design of Concrete Structures*, 12th ed., McGraw-Hill, New York, 1997.

CHAPTER 5:
PHASE 3 BENT TESTS

5.1 INTRODUCTION

Phase 3 consisted of the design, construction, and testing of a trestle pile bent and a column bent. Construction and testing of the bents were conducted at the construction yard of Champagne-Webber, Inc. in Houston, Texas. The primary objective of Phase 3 was to assess the constructability of the connection alternatives developed in Phases 1 and 2, including single-line and double-line grout pockets, grouted vertical ducts, and bolted connections. A secondary objective of Phase 3 was to examine connection and bent behavior at service-level and factored-level vertical loads, as well as to observe failure modes for the pile bent.

Figures 5.1 and 5.2 show fabrication and the test setup of the trestle pile and column bents, respectively. Construction included fabrication of precast caps, footings with stub columns or piles, cap placement, connection grouting, and loading. An important aspect of Phase 3 was the close interaction between UT, TxDOT and Champagne-Webber. UT designed the bents with TxDOT input and review, TxDOT produced plan sheets with UT review; and Champagne-Webber carried out construction with some UT and TxDOT guidance.



a. Pile Bent Construction *b. Pile Bent Test Setup*

Figure 5.1 Phase 3 Pile Bent

Phase 3 enabled the following new issues to be investigated:

- Contractor construction of full-scale bent caps and connections with feedback for all stages of the construction sequence
- Contractor use of plan sheets and grout specification
- Construction tolerances for multiple connections
- Shim stability during cap setting and adequacy under loading
- Suitability of plate and leveling nut option for cap setting
- Trial grout batches
- Grouting of pile and column ducts
- Contractor-engineer interaction before and during grouting
- Grouting of multiple connections

- Grouting under adverse weather conditions
- Multiple connection and bent behavior under loading.

a. Column Bent Construction *b. Column Bent Test Setup*

Figure 5.2 Phase 3 Column Bent

By addressing these issues, Phase 3 significantly helped to refine and validate connection details developed during earlier research stages.

This chapter summarizes Phase 3, with the main emphasis on constructability. Important details of Phase 3 are summarized, including the test matrix, specimen design, specimen fabrication, cap setting, and grouting. Contractor feedback for each stage of construction is provided, and the lessons learned during construction are summarized. Finally, the instrumentation, test setup, and test procedure are reviewed, and test results are discussed.

5.2 SYNOPSIS OF TEST PROGRAM

At the initial construction meeting, the contractor was given a copy of the plan sheets, the precast connection specification, a summary of the lessons learned from Phase 2 grouting with photos, and a sketch of the test setup. Limited materials were also sent to the contractor, including ducts and connectors. Based on this input, the contractor successfully built two bents with precast caps. Interaction between the contractor and the research team was minimal during fabrication and cap setting, but necessarily increased during grouting operations and testing.

5.2.1 Test Matrix

5.2.1.1 Background

Table 5.1 shows the test matrix for the trestle pile and column bents. The intent of Phase 3 construction was to investigate constructability issues for connection alternatives. With a few variations, connection types matched those used in Phase 2, as Phase 2 testing indicated suitable behavior for all connection types. Although the use of different connection types in a single bent is not expected in practice, each bent included a vertical duct, grout pocket, and bolted connection to maximize contractor experience in fabrication and grouting of individual connection types while still maintaining a reasonable scope and cost for Phase 3. In most cases, minimum provisions suggested by Phase 1 and 2 tests were used to proportion Phase 3 specimens to test the adequacy of those provisions. For example, a minimum number of connectors and minimum embedment depths were used. The following sections briefly explain the use of the parameters summarized in the test matrix. Loading is discussed in Section 5.2.1.9.

Table 5.1 Phase 3 Test Matrix

Bent ID	Connection			embedmt. (in)	Grout		Shims
	type	connectors	confinement		brand	method	
PVDE	vertical duct embedded	2/2 straight #9's @ 15 in. embedment	#3 @ 4 in. 18-in. diameter	15	MF928	tremie tube	two 3 in. x 3 in. plastic shims
PSL	single-line grout pocket	1/3 straight #9's @ 15 in. embedment	#3 @ 4 in. 18-in. diameter	15	EHF	bucket	none
PSLE	single-line grout pocket embedded	1/3 straight #9's @ 15 in. embedment	#3 @ 4 in. 18-in. diameter	15	EHF	bucket	none
PBC	bolted	2/2 1-in. diameter bolts entire depth	#3 @ 4 in. 18-in. diameter	15	Sika 212	tremie tube	two 3 in. x 3 in. plastic shims
CVD	vertical duct	2/2 straight #9's @ 15 in. embedment	#3 @ 4 in. 24-in. diameter	15	MF928	tremie tube	one 3 in. x 3 in. steel shim
CDL	double-line grout pocket	2/2 straight #9's @ 15 in. embedment	#3 @ 4 in. 24-in. diameter	15	Sika 212	bucket	none
CBC	bolted	2/2 1-in. diameter bolts entire depth	#3 @ 4 in. 24-in. diameter	15	EHF	tremie tube	two 5 in. x 5 in. leveling plates

EHF=Euclid Hi-Flow, neat; Sika 212=Sika 212, neat; MF928=Masterflow 928, neat

5.2.1.2 Bent Type and Dimensions

Two bent types were constructed, a trestle pile bent and a column bent. Plan sheets are shown in Figures 5.3 and 5.4. To simplify construction requirements and enable formwork reuse for the two bents, a single bent cap size of 2 ft.-6 in. x 2 ft.-9 in. x 25 ft. was selected for both bents. The selected cross section allowed comparison with Phase 2 results. The cap length of 25 ft. allowed a realistic pile spacing of 6.5 ft. as well as realistic simulation of girder loading using rams. Three connections were used in the column bent, resulting in very short 9-ft spans. However, this was considered acceptable, as constructability was the primary objective of Phase 3. In addition, excessively large vertical loads would have been required to produce failure of the column bent, even with considerably longer spans.

Pile and column sizes also matched those selected in Phase 2. The 2-ft. high pile and column stubs were again selected to facilitate grouting operations and testing. However, this did not significantly affect the quality of the contractor's experience. In lieu of a test frame, special details were used in the footings to resist vertical loading. See Section 4.2.1.2 for more detail regarding the rationale of the bent type and dimensions.

5.2.1.3 Connection Type and Configuration

Table 5.1 lists the connection types used in Phase 3. In addition to a vertical duct (embedded pile) and a bolted connection (PVDE and PBC), the pile bent included two single-line grout pocket connections, one surface-flush (PSL) and the other with a two-inch embedment above the bottom surface of the bent cap (PSLE). PSL provided a direct comparison to PSLE as well as a nearly direct comparison to the Phase 2 PSL connection. The column bent included each of the connection types investigated in Phase 2. The primary difference between Phase 2 and Phase 3 column connections was the use of a plate-and-leveling-nut option in cap setting for the Phase 3 bolted connection (CBC). Embedment alternatives were only used for the pile bent. The embedment approach was essentially the same as that used in the Phase 2 PSL specimen, in which a 3.5-in. deep void in the bottom of the cap was formed to house the top of the pile.

As shown in Figures 5.3 and 5.4, connectors were configured in the same pattern as Phase 2 specimens. In addition, the same tapers and tolerances were used for single-line and double-line grout pockets (see Table 4.2).

5.2.1.4 Connector Size, Anchorage, and Embedment Depth

Connector sizes, anchorage and embedment depths were the same as for Phase 2 specimens.

5.2.1.5 Confining Reinforcement

Phase 2 tests revealed minimal cracking in the connection region, even at failure. Thus, confining reinforcement was not expected to contribute significantly. Nevertheless, it was decided to use confining reinforcement in Phase 3 tests as a conservative and inexpensive measure. To simplify construction, the height of spiral rebar was extended the full depth between top and bottom layers of longitudinal reinforcement for all connections.

5.2.1.6 Grout

To verify grout characteristics, the three grout brands tested in Phases 1 and 2—Masterflow 928 (MF928), Euclid High Flow (EHF), and Sika 212—were again used in Phase 3. Grouts were used in three different stages of grouting operations—trial batches, grouting of connectors into the piles and columns, and grouting of the connections. All grout brands were used in each stage.

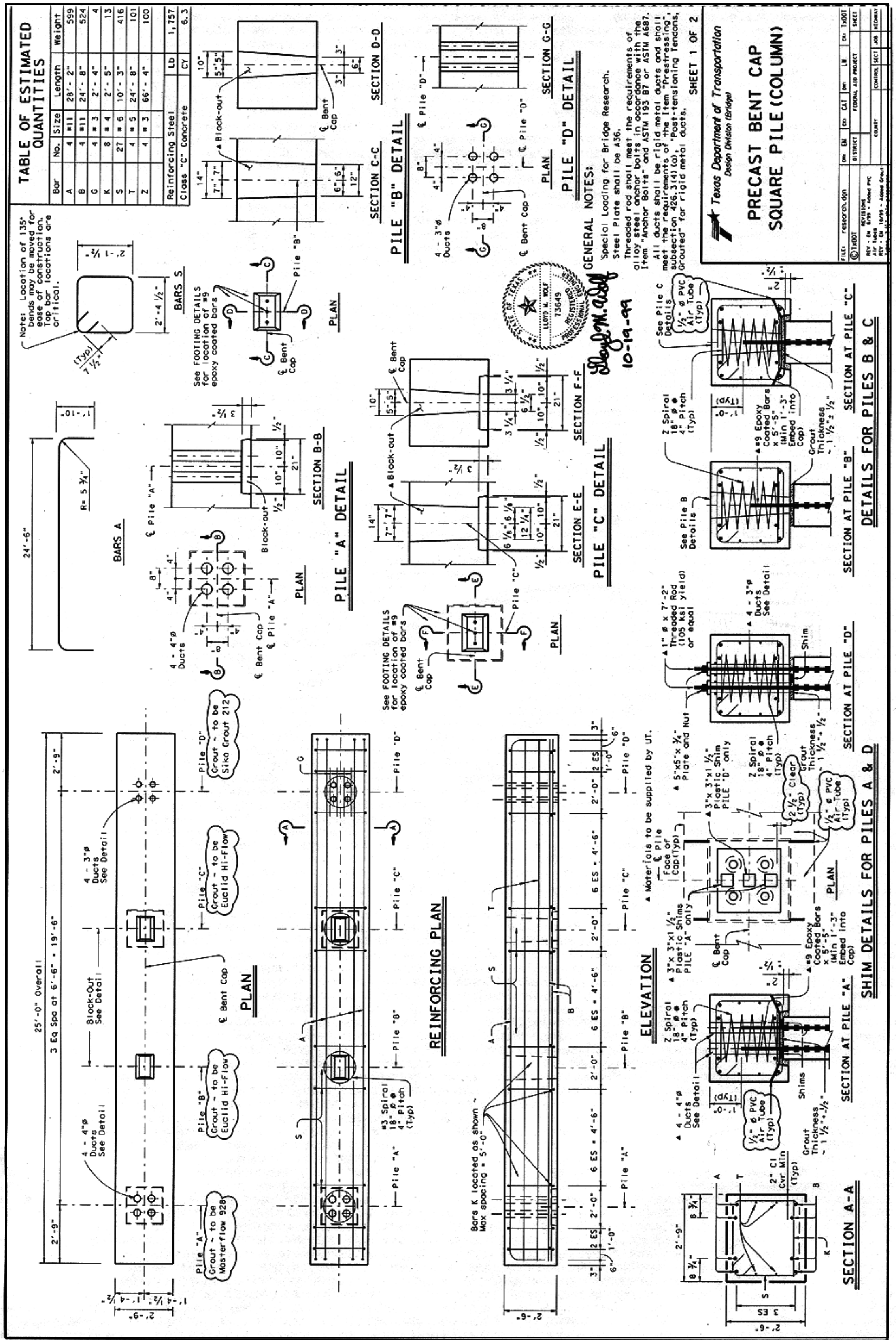


Figure 5.3 Plan Sheet—Trestle Pile Bent

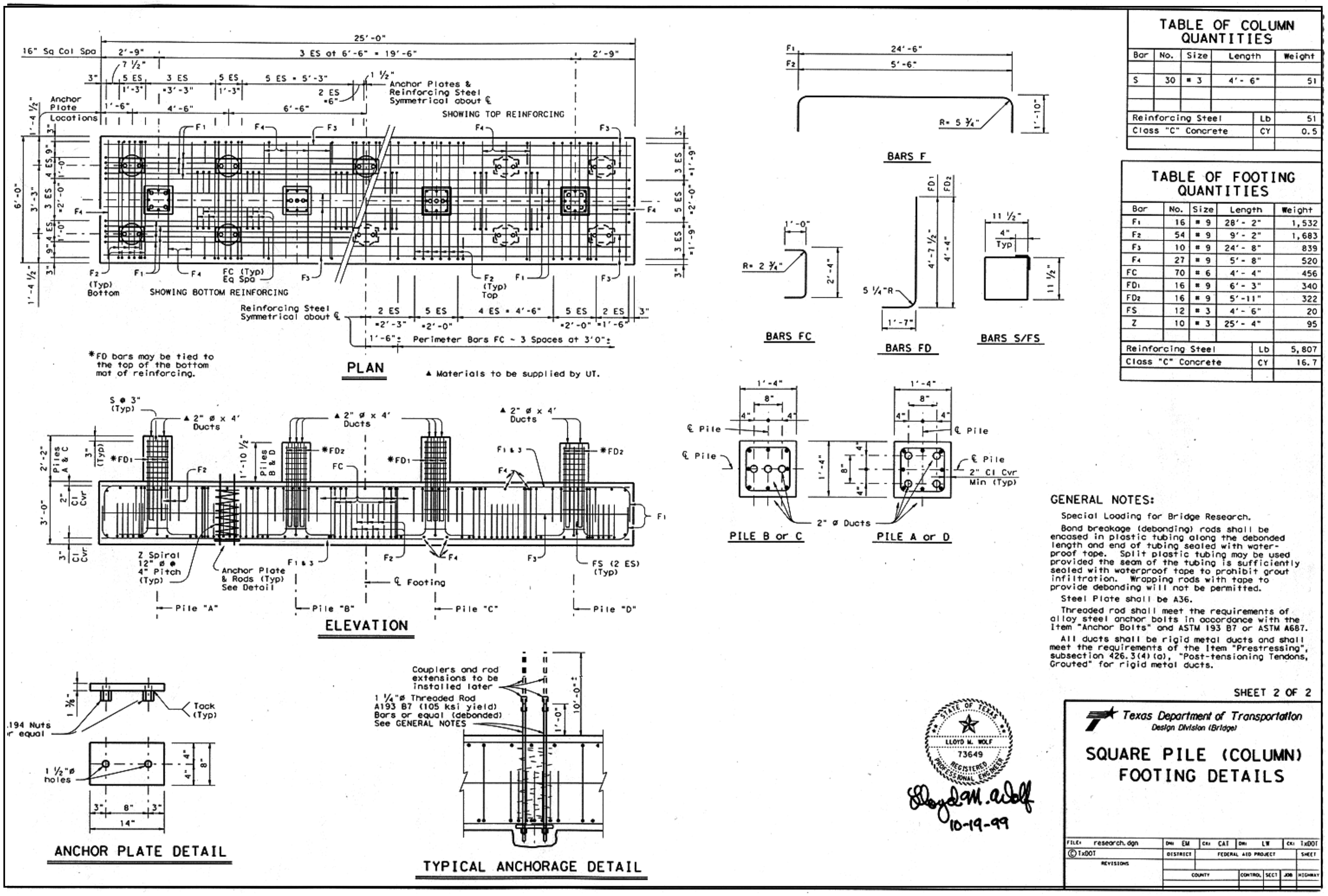


TABLE OF COLUMN QUANTITIES

Bar	No.	Size	Length	Weight
S	30	# 3	4' - 6"	51
Reinforcing Steel			Lb	51
Class "C" Concrete			CY	0.5

TABLE OF FOOTING QUANTITIES

Bar	No.	Size	Length	Weight
F₁	16	# 9	28' - 2"	1,532
F₂	54	# 9	9' - 2"	1,683
F₃	10	# 9	24' - 8"	839
F₄	27	# 9	5' - 8"	520
FC	70	# 6	4' - 4"	456
FD₁	16	# 9	6' - 3"	340
FD₂	16	# 9	5' -11"	322
FS	12	# 3	4' - 6"	20
Z	10	# 3	25' - 4"	95
Reinforcing Steel			Lb	5,807
Class "C" Concrete			CY	16.7

GENERAL NOTES:

Special Loading for Bridge Research.

Bond breakage (debonding) rods shall be encased in plastic tubing along the debonded length and end of tubing sealed with waterproof tape. Split plastic tubing may be used provided the seam of the tubing is sufficiently sealed with waterproof tape to prohibit grout infiltration. Wrapping rods with tape to provide debonding will not be permitted.

Steel Plate shall be A36.

Threaded rod shall meet the requirements of alloy steel anchor bolts in accordance with the Item "Anchor Bolts" and ASTM 193 B7 or ASTM A687.

All ducts shall be rigid metal ducts and shall meet the requirements of the Item "Prestressing", subsection 426.3(4)(a), "Post-tensioning Tendons, Grouted" for rigid metal ducts.

SHEET 2 OF 2

Texas Department of Transportation
Design Division (Bridge)

SQUARE PILE (COLUMN)
FOOTING DETAILS

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73649

Lloyd M. Wolf
10-19-99

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Figure 5.3 (cont.): Plan Sheet—Trestle Pile Bent

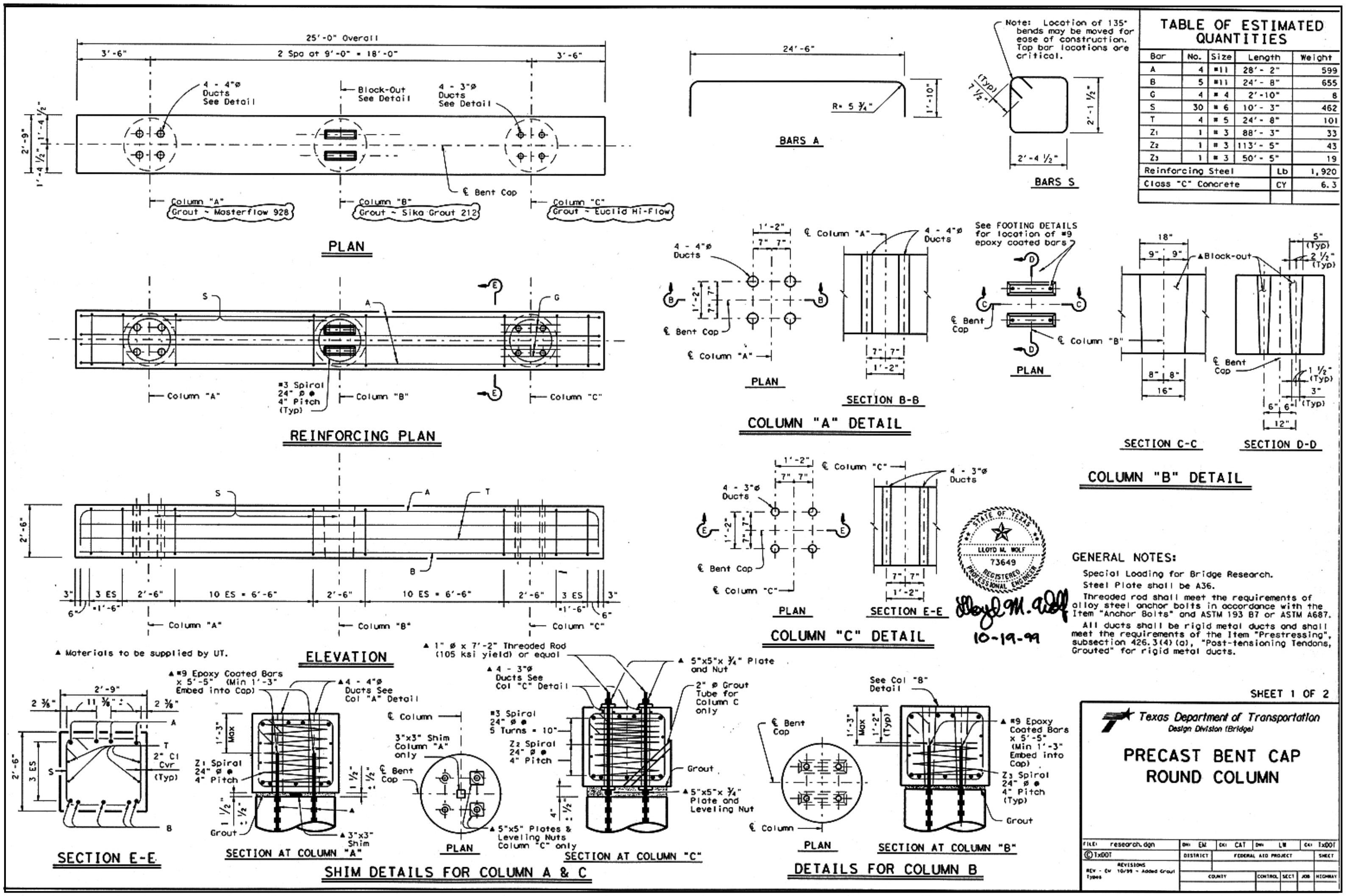


TABLE OF ESTIMATED QUANTITIES

Bar	No.	Size	Length	Weight
A	4	#11	28'- 2"	599
B	5	#11	24'- 8"	655
G	4	# 4	2'-10"	8
S	30	# 6	10'- 3"	462
T	4	# 5	24'- 8"	101
Z1	1	# 3	88'- 3"	33
Z2	1	# 3	113'- 5"	43
Z3	1	# 3	50'- 5"	19
Reinforcing Steel			Lb	1,920
Class "C" Concrete			CY	6.3

GENERAL NOTES:

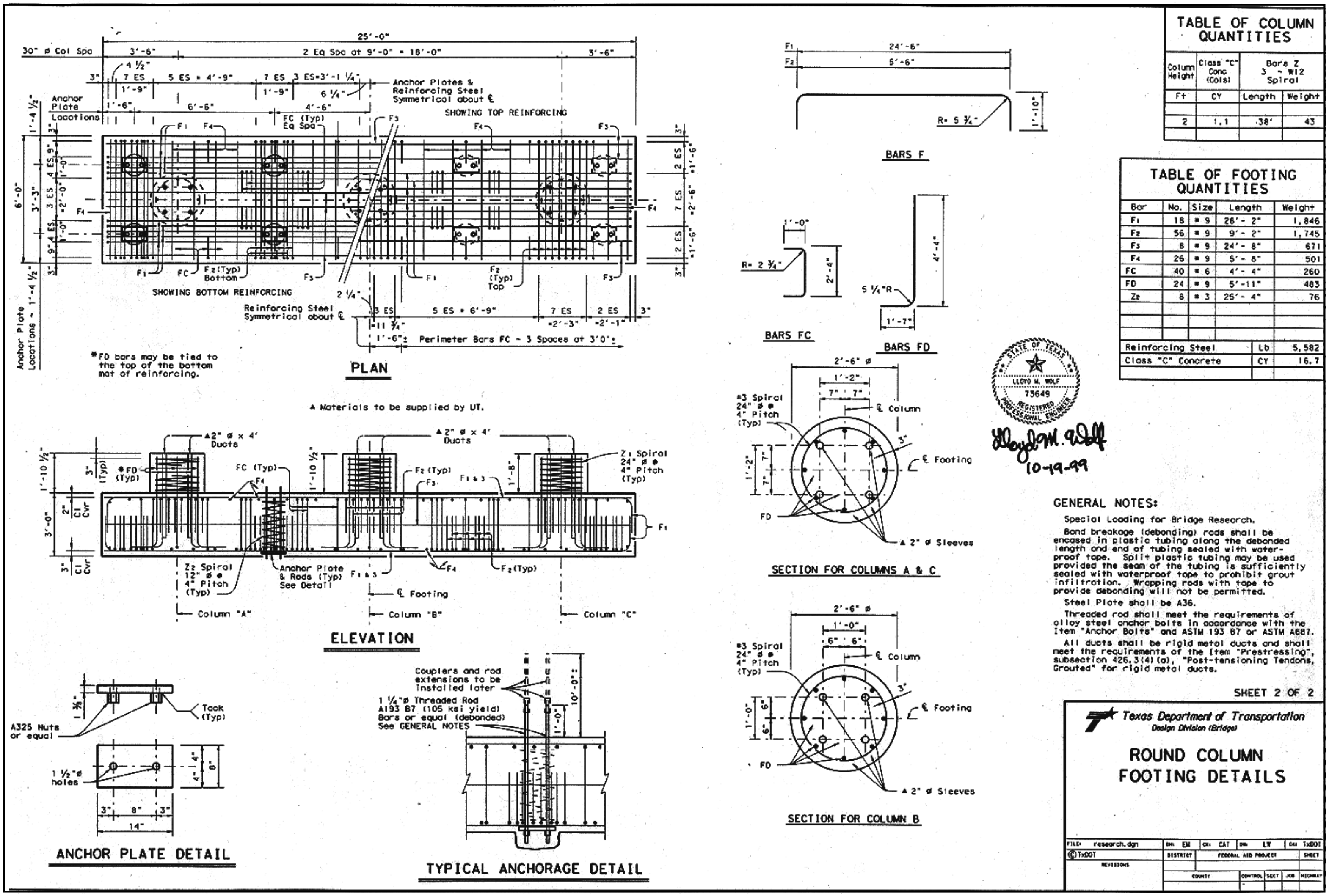
Special Loading for Bridge Research.

Steel Plate shall be A36.

Threaded rod shall meet the requirements of alloy steel anchor bolts in accordance with the Item "Anchor Bolts" and ASTM 193 B7 or ASTM A687.

All ducts shall be rigid metal ducts and shall meet the requirements of the Item "Prestressing", subsection 426.3(4)(a), "Post-tensioning Tendons, Grouted" for rigid metal ducts.

Figure 5.4 Plan Sheet—Column Bent



*FD bars may be tied to the top of the bottom mat of reinforcing.

▲ Materials to be supplied by UT.

TABLE OF COLUMN QUANTITIES

Column Height	Class "C" Conc (Cols)	Bars Z 3 ~ W12 Spiral	
Ft	CY	Length	Weight
2	1.1	·38'	43

TABLE OF FOOTING QUANTITIES

Bar	No.	Size	Length	Weight
F₁	18	# 9	28'- 2"	1,846
F₂	56	# 9	9'- 2"	1,745
F₃	8	# 9	24'- 8"	671
F₄	26	# 9	5'- 8"	501
FC	40	# 6	4'- 4"	260
FD	24	# 9	5'-11"	483
Z₂	8	# 3	25'- 4"	76
Reinforcing Steel			Lb	5,582
Class "C" Concrete			CY	16.7

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73649

Lloyd M. Wolf
10-19-99

GENERAL NOTES:

Special Loading for Bridge Research.

Bond breakage (debonding) rods shall be encased in plastic tubing along the debonded length and end of tubing sealed with waterproof tape. Split plastic tubing may be used provided the seam of the tubing is sufficiently sealed with waterproof tape to prohibit grout infiltration. Wrapping rods with tape to provide debonding will not be permitted.

Steel Plate shall be A36.

Threaded rod shall meet the requirements of alloy steel anchor bolts in accordance with the Item "Anchor Bolts" and ASTM 193 B7 or ASTM A687.

All ducts shall be rigid metal ducts and shall meet the requirements of the Item "Prestressing", subsection 426.3(4)(a), "Post-tensioning Tendons, Grouted" for rigid metal ducts.

SHEET 2 OF 2

Texas Department of Transportation
Design Division (Bridge)

ROUND COLUMN
FOOTING DETAILS

FILE: research.dgn
© TxDOT
REVISIONS
DN: EM | CK: CAT | DW: LW | CK: TxDOT
DISTRICT | FEDERAL AID PROJECT | SHEET
COUNTY | CONTROL | SECT | JOB | HIGHWAY

Figure 5.4 (cont.): Plan Sheet—Column Bent

MF928 was used for the more critical function of tremie-tube grouting the vertical duct connections for each bent (PVD and CVD). It was expected that MF928 would work successfully for any connection type. Although EHF demonstrated lower strength and an unusually porous top surface in Phase 2, EHF was used for single-line grout pockets for comparison with Phase 2 results. EHF was also used for the less critical function of filling the CBC ducts. Sika 212 was used for the double-line grout pocket specimen (CDL) as a direct comparison to the Phase 2 CDL connection, which exhibited a gap at the bedding layer. This was to assess potential segregation of Sika 212, as observed in Phase 2. Sika 212 was also used for filling the ducts of the PBC connection. The contractor obtained grouts locally.

5.2.1.7 Grouting Methods

As for Phase 2, bucket and tremie-tube grouting were used. The bucket approach was used for grout pocket connections, whereas tremie-tube grouting was used for vertical duct and bolted connections. Pumping grout was considered as an alternative to gravity-flow grouting, but was not used.

5.2.1.8 Shims

Based on successful use of both plastic and steel shims in Phase 2 tests, plastic and steel shims were used in Phase 3. A primary difference was the number of shims used at a pile or column top. Three-point bearing at exterior piles or columns was considered sufficient. The pile bent used two plastic shims at the PVDE pile and one plastic shim at the PBC pile (Figure 5.3). The round column used a steel shim at the PVD column in combination with two steel plates and leveling nuts at the PBC column. To facilitate grouting at the bedding layer, shims were placed no closer than 2.5 in. from pile or column faces.

5.2.1.9 Loading

To maintain a reasonable scope for field testing, only vertical loads were applied to the bents. As shown in Figures 5.1b and 5.2b, a series of rams were positioned on both sides of connections. Transfer girders were placed over rams and tied into the footing using threaded rods. Based on assumptions of a typical superstructure and AASHTO provisions, dead and live load combinations corresponding to service and factored levels were applied to the bent. The use of multiple pumps and rams allowed application of transverse and/or longitudinal moments (M_{trv}, M_{long}) to be investigated. After proof testing connections to realistic service and factored levels, "failure" loads and eccentricity combinations were applied to the bent. Although maximum loads and eccentricities were limited by the capacity of the testing system and test setup, connection failure was predicted for the pile bent. Figures 5.5 and 5.6 show the configuration of rams and load combinations used for the pile and column bents, respectively. Corresponding eccentricities and axial load-moment (P-M) combinations are described in subsequent sections.

5.2.2 Specimen Design

This section briefly summarizes key aspects of the design of each bent, which became the basis for the plan sheets shown in Figures 5.3 and 5.4. Bent configurations were defined based on the considerations mentioned in the previous section. Each bent was designed for dead and live loads associated with the AASHTO Standard Specifications. The load cases described in Section 5.2.1.9 were used to determine moment and shear envelopes with corresponding axial loads. 2-D frame analyses assuming rigid connections were conducted to determine in-plane design actions. Although smaller bent cap moments and larger connection moments were assumed in design than actually developed for the transverse direction, this did not adversely affect testing. Simultaneous moments in the longitudinal direction were determined assuming equal stiffness for all connections and columns. Connections and columns were assumed to be rigid in calculating torsion. The cap design did not require unusual amounts of flexural or shear reinforcement.

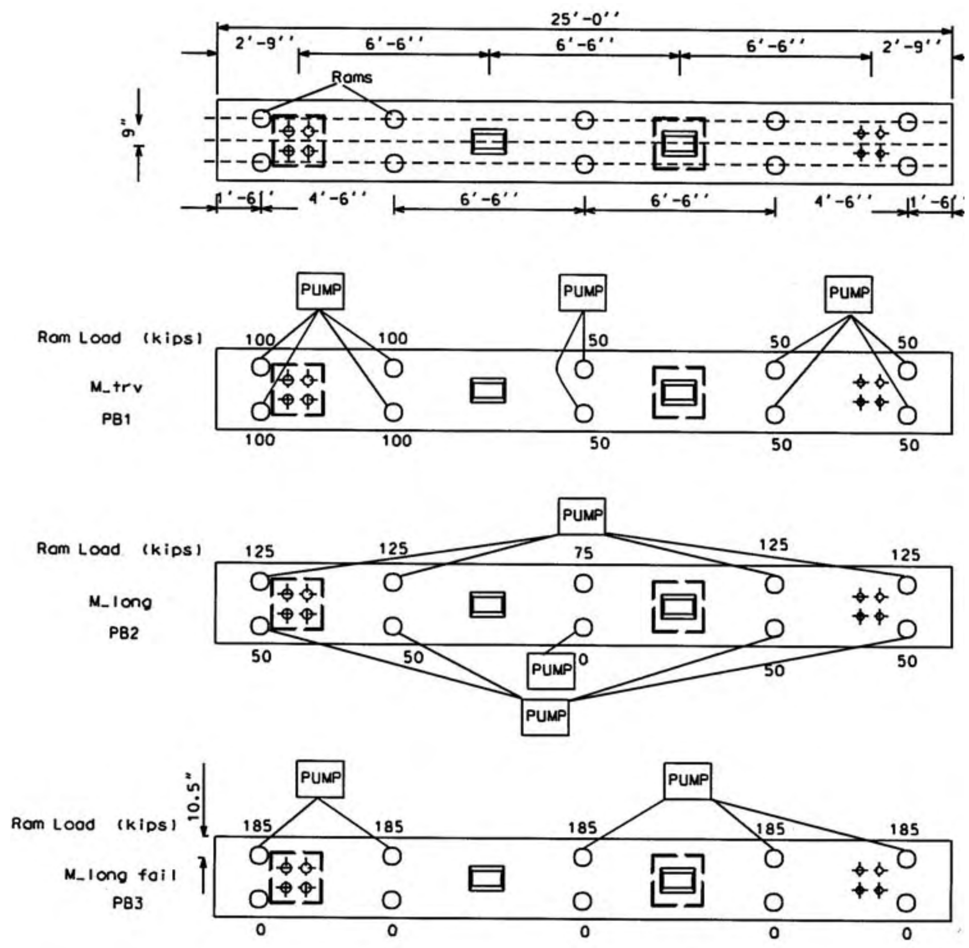


Figure 5.5 Ram Configuration and Load Level—Trestle Pile Bent

Connection types were selected based on Phase 1 and 2 results. Capacities for connector arrangements were based on P-M strength interaction diagrams and the Reciprocal Load Method [4.5]. The configurations closely matched those used in Phase 2 tests and allowed for comparison. The one disadvantage of using the small size and number of connectors for the bents was that bar and duct spacing were relatively large and thus anchorage of bars and ducts was not so severely challenged. The 15-inch embedment depth for grout pocket and vertical duct connections was based on Phase 1 and 2 results, which indicated an embedment of $13d_b$ was sufficient to achieve bar yield for a concrete strength of 5.4 to 6.0 ksi. In addition, shear friction capacity was checked. No explicit calculation was performed for spiral confining reinforcement for grout pocket and vertical duct connections. Reduced spacing of spiral reinforcement and extra reinforcement in the anchorage region at the cap top for bolted connections were based on a conservative estimate of bursting and spalling stresses.

5.2.3 Specimen Fabrication

For each bent, the contractor constructed a bent cap and a footing with piles or columns. Pile and column stubs were cast monolithically with the footing. The following sections briefly discuss the fabrication of the bents. Contractor comments for each stage of construction are summarized in Table 5.2.

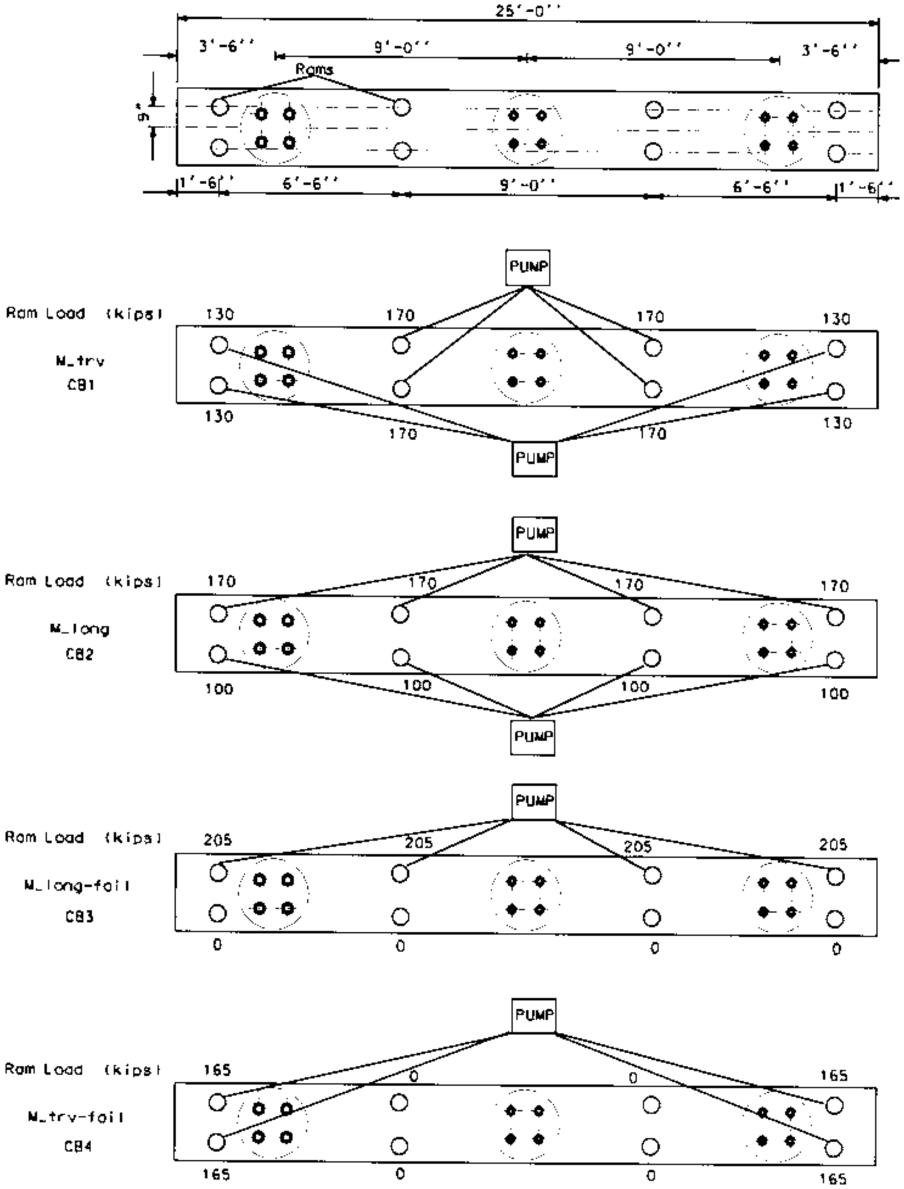


Figure 5.6 Ram Configuration and Load Level—Column Bent

5.2.3.1 Bent Caps

Figure 5.7 shows the steel forms used for the pile cap. A similar form was used for the column cap. Grout pocket inserts and ducts for vertical duct and bolted connections are visible along the cap. Figure 5.8a shows the single-line grout pocket insert together with the spiral extending to the top of the longitudinal rebar. Plywood was used to stabilize the forms during casting. Grout pocket forms were easily removed with a crow bar after casting. Figure 5.8b shows a plan view of the grout pocket insert, including a 2x4 placed inside the insert for stabilization. Inserts were tied to the bottom form using Hilti fasteners.

Table 5.2 Contractor Comments

Cap Fabrication

1. No major difficulties occurred during cap fabrication.
2. Use steel grout pocket forms for repeatability on large jobs. They are reusable, don't warp, bolt on easily, and are simple and quick to remove.
3. Both single-line and double-line grout pockets were easy to form.
4. Major limitation to grout pocket size or shape is placement of longitudinal rebar.
5. Styrofoam was not used in forming grout pockets because it is too delicate for handling, hard to control, and tends to float during casting. Steel or plywood is better.
6. Forming the base void for embedment of piles was simple.
7. Placement of confining spiral was simple. Spirals can be compressed, placed in form, then expanded. Welded wire fabric cage would be much more difficult to place.
8. Grout pocket connections were easier to fabricate than connections with ducts. They were more forgiving related to placement.

Footing Fabrication

1. Detail a minimum gap between ducts and vertical pile or column bars (e.g., 0.75 in.). A conflict arose between duct placement and pile vertical bars during fabrication.
2. Ducts in piles or columns should provide a tolerance of at least +/-1 in. for placement of connectors.
3. Accurate positioning of ducts in columns required the use of a steel template (plate). In some cases, the template was used at both the top and bottom of the ducts, requiring that the template be cast into the column.
4. Placement of confining spiral around ducts was simple.

Casting

1. Typical casting and vibrating procedures were used in casting.
2. Adequate spacing between pile ducts was provided (2 in.).
3. This project was limited in scope. Profitability is defined by speed and repeatability. Projects with repeatability are attractive.

Table 5.2 Contractor Comments (Cont.)

Cap Setting

1. Plastic shims deformed and shifted some during cap setting. Steel shims are suggested to prevent this. Plastic shims were moved 1 in. to provide level bearing. Two shims per column is recommended, even if one doesn't bear. Roughened recess (1/4 in.) at column top may help prevent movement of shims.

2. Other options are available for cap setting in addition to shims: 1) friction collars; 2) shims and friction collars; 3) through-bolt with jacks to support cap, with or without shims; and 4) plates and leveling nuts.

3. Plates with leveling nuts on the bolted connection worked very well. This option provided adjustability and would provide cap stability as well.

4. Ducts in the cap should be made as large as possible for horizontal tolerances. Tolerances provided were acceptable, but 3 in. ducts with 1 in. bolt (+/- 1 in.) is the minimum acceptable tolerance. Ducts in footing should not be less than 2 in. when using #9 bars.

5. No difficulties occurred in setting the cap over multiple connectors. Guide pipes are a good idea for ducts.

6. No significant constructability difference resulted in embedding piles into cap versus surface flush.

7. If special measures are necessary for cap stability in the field, friction collars or through-bolts may be used. Bolted connections directly provide stability.

8. Grouted sleeve couplers would not be recommended due to excessively tight tolerances.

Trial Batch

1. Foreman and TxDOT Materials representative should discuss specification prior to trial batch.

2. Trial batch is important for this new system. Adjustments to grout mixing were made and grouts were easily compared.

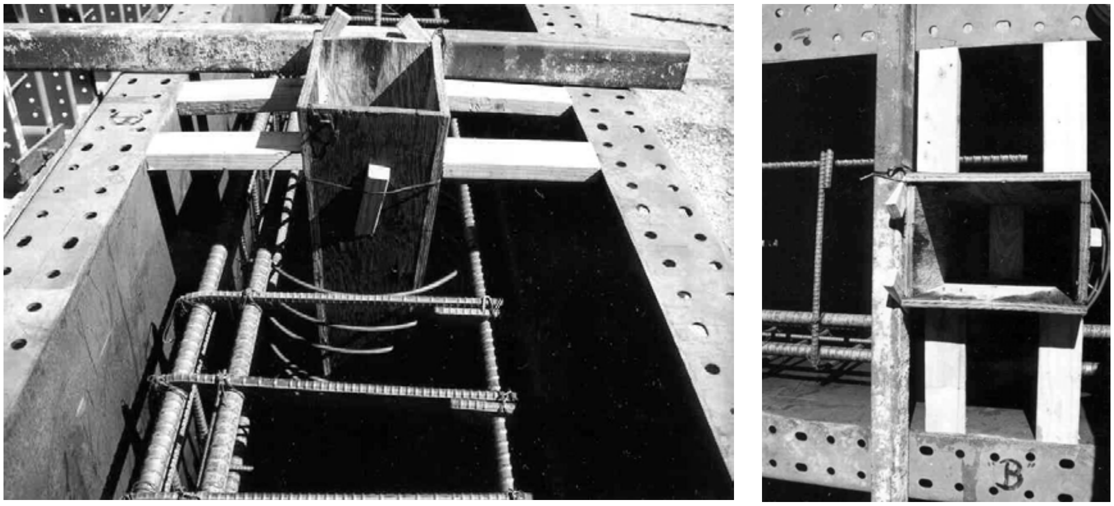
3. Trial batch should be conducted 1-2 weeks before actual grouting to ensure lessons learned are implemented and an acceptable grout is selected.

Table 5.2 Contractor Comments (Cont.)

Grouting

1. Grouting should use same weather restrictions as casting concrete.
2. Symons fiberglass formwork for round column was more watertight than wood used with pile. Round form did not leak after caulking. Symons was also willing to fabricate form for square pile.
3. Prewatering is important to identify leaky formwork. Prewatering for longer duration (e.g., 4-6 hours) is possible, but requires care in making formwork watertight. A quick drying caulk or 2-part epoxy may help plug leaks.
4. Twenty minute pre-grouting meeting with TxDOT representative is strongly recommended to ensure correct operations are performed. Step-by-step procedures should be reviewed.
5. Step-by-step grouting procedures should be listed in plans.
6. TxDOT representative should be available for interaction, especially during initial grouting operations. However, once the representative sees that the contractor knows how to carry out operations, he should stay clear.
7. Connections can be consolidated using a tamping rod (rebar). A vibrator is not necessary.
8. Grouting operations for a typical 3-column bent would require a six-man crew plus a crane operator: three guys on the ground mixing, one man in basket, two on cap. After all grout is mixed (10 minutes), man-basket is used to lift grout to cap top. Bucket grouting would be easier than continuous tremie-tube grouting. Air vents should be on top or side of cap for easy access for workers on cap to plug. Workers on ground mix additional grout while others grout connection.

Figure 5.7 Pile Cap Formwork

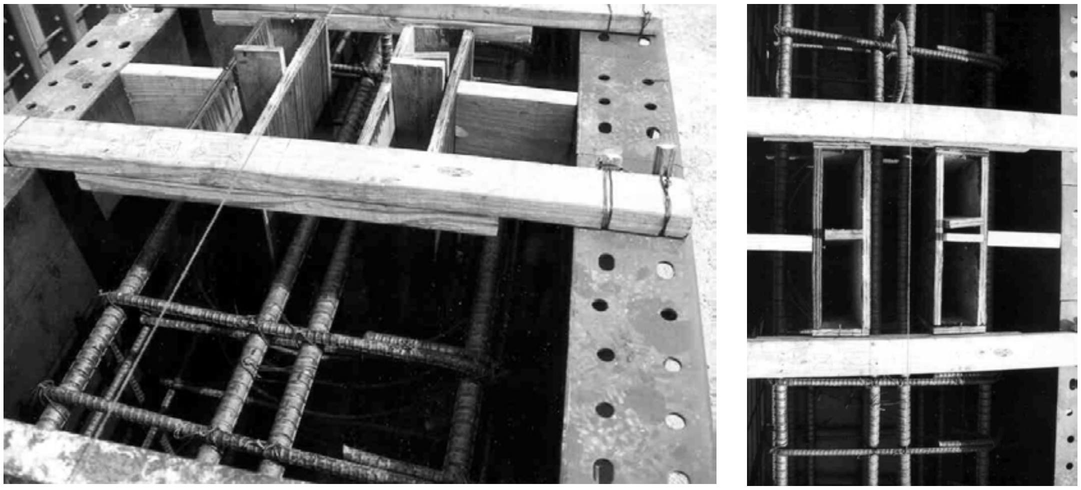


a. Side View *b. Plan View*

Figure 5.8 Single-line Grout Pocket Insert

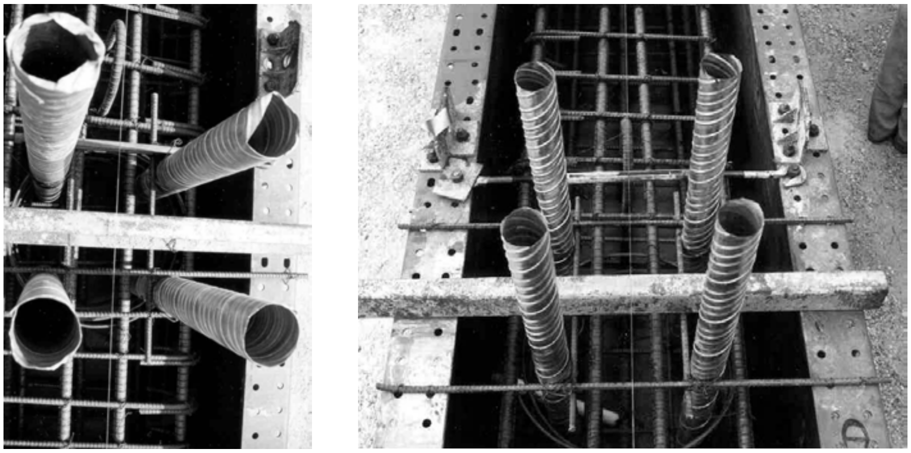
Figure 5.9 demonstrates the forming of double-line grout pockets. Note that clearance is provided for longitudinal bars between pockets. Figures 5.10a and 5.10b show the configuration of embedded ducts used for the vertical duct and bolted connections, respectively. Auxiliary rebar was necessary to position and stabilize ducts for casting. A 2-in. grout tube is shown at the bottom of the formwork in Figure 5.10b.

Fabrication of the caps was fairly simple and straightforward. The contractor reported that grout pockets were easier to fabricate than vertical duct or bolted connections, due to greater forgiveness in placement of inserts (see Table 5.2). The contractor also said that wood inserts were easy for forming grout pockets, but that steel forms would be more advantageous for jobs with significant repeatability. Placement of spiral confinement was simple. Spiral rebar was preferred much more than welded wire fabric due to ease of placement.



a. Side View *b. Plan View*

Figure 5.9 Double-line Grout Pocket Inserts



a. Vertical Duct Connection *b. Bolted Connection*

Figure 5.10 Embedded Ducts

5.2.3.2 Footings with Stub Piles or Columns

The rebar cage for the footing and pile stubs is shown in Figure 5.1a. For convenience, rebar and ties were used in piles rather than strands and spiral. Ducts used to house connectors are shown extending above the piles. Ducts were tied to the vertical pile bars (Figure 5.11). Threaded rods and spiral confinement used for loading the bent are shown on both sides of the stub pile. Figure 5.2a shows a template that was used to align column ducts. In some cases, a second template was used at the base of column stubs for alignment (Figure 5.12).

Although vertical bars provided a convenient support for pile and column ducts, during construction a vertical bar interfered with placement of a column duct. The contractor subsequently recommended a minimum ¾-in. gap to prevent this. In addition, since placement of connectors is particularly critical, ducts with a tolerance larger than +/-0.5 in. was suggested. The use of templates shown in Figure 5.12 may hinder casting and vibrating operations. Two alternatives are:

- Use templates with a central opening to facilitate casting. Fabricate ducts and templates as one assembly and place the assembly into the column form after the column has been cast within 5 to 6 feet of the top. The last portion would be cast using a lay-down bucket.
- Tie ducts to supplementary crossbars that would be tied to the column cage. Clamping or tack welding to supplementary bars could be used, although this approach may not be as reliable as using templates.

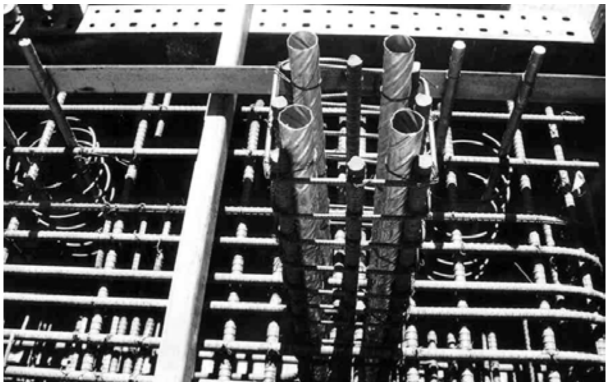


Figure 5.11 Ducts within Pile

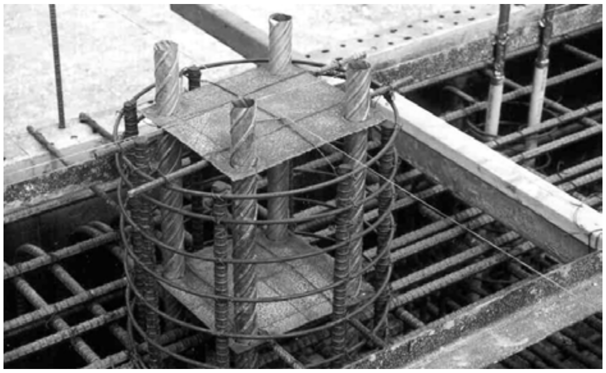


Figure 5.12 Templates used to Align Ducts

5.2.3.3 Casting

Typical procedures were used in casting and curing the bent components. Unlike Phase 2, three casting operations were used, one for each of the following: caps, footings, and stub piles and columns. Casting of the footings is shown in Figure 5.13. Figure 5.14 shows a single-line grout pocket in the pile cap after form removal.

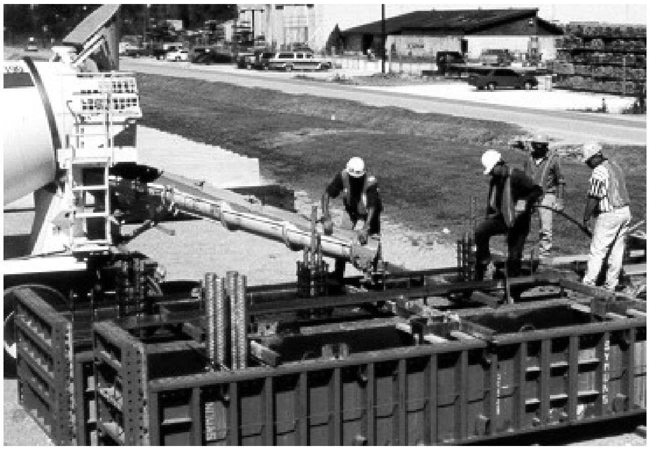


Figure 5.13 Casting of Bent Caps

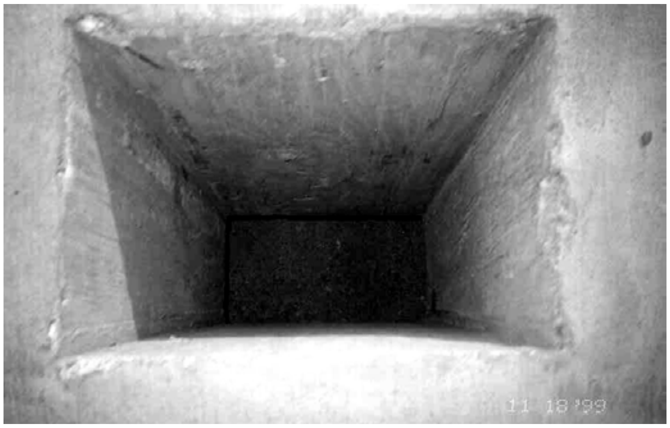


Figure 5.14 Single-line Grout Pocket after Form Removal

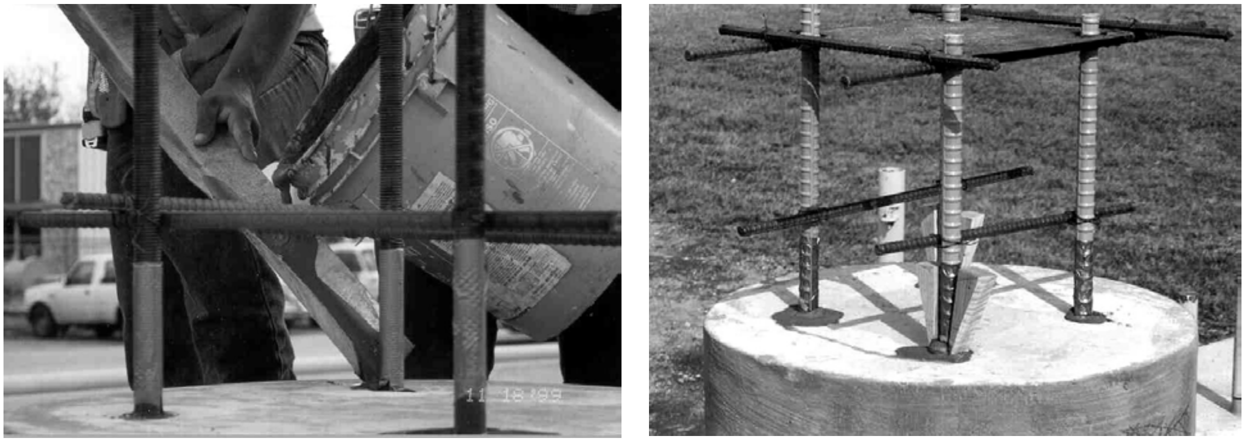
5.2.3.4 Grouting of Connectors into Piles and Columns

Following form removal, connectors were grouted into the pile and column ducts. The same grout brands that were specified for the cap connections were also used for grouting these ducts. Grout flow and strength are listed in Table 5.3. Grout was prepared in the same way as trial batches, which are discussed in Section 5.2.5.1. Gravity-flow grouting of column ducts is shown in Figure 5.15a. An open funnel aided bucket grouting. The contractor felt that tolerances at least ¾-in. to 1-in. should be provided. This is important since tight construction tolerances were achieved because piles were cast in place rather than precast. Figure 5.15b shows the wooden shims used to maintain verticality of the connectors after grouting.

Table 5.3 Grout Flow and Strength for Grouting Connectors into Columns and Piles

Grout Brand	Water (lbs/bag)	Temp (F)	Flow (sec)	Grout Cube Strength (ksi)
EHF	9.6	77	30	4.6 (21-day, test day-1)
Sika 212	8.9	77	20	5.4 (21-day, test day-1)
MF928	10.0	77	29	8.1 (21-day, test day)

Figure 5.16 shows the footings with pile and column stubs after form removal and grouting of connectors. Figure 5.16a shows the 2/2 and 1/3 connection configurations for the #9 epoxy-coated bars, as well as the two pile-embedment options (surface-flush vs. embedded). The bolts shown in the background of the figure extend through the cap and are therefore much taller than the other connectors. The connectors in the 1/3 option were cut to the appropriate length after casting. The PVC extending from the footing housed threaded rods for testing. Figure 5.16b shows the connectors for the column bent after form removal.



a. Gravity-flow Grouting *b. Shims*

Figure 5.15 Grouting of Connectors into Columns

a. Piles Bent *b. Column Bent*

Figure 5.16 Bents after Casting of Concrete and Grouting of Connectors

5.2.4 Cap Setting

Although a number of different connection types were used within each bent, Phase 3 still provided a realistic cap setting experience for the contractor. Different variables were investigated for each bent. The trestle pile bent used plastic shims in three-point bearing, while the column bent used a steel shim at one column plus plates and leveling nuts at the opposite column.

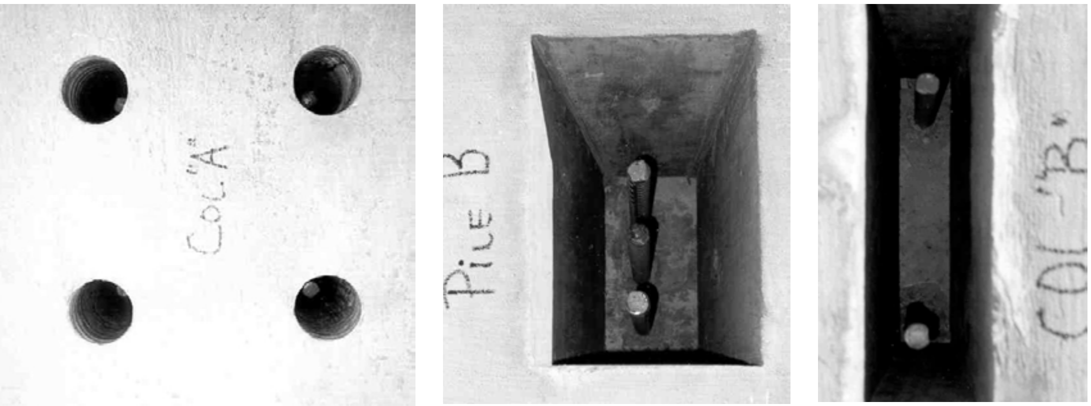
Figure 5.17 shows cap setting for the pile bent connections. Plastic shims appeared to deform and shift some during cap setting, requiring shims to be moved ~1 in. to provide level bearing. No difficulties were experienced in setting the cap over multiple connections or for embedded options. Ducts in the cap provided sufficient tolerance, although the contractor commented that 3-in. ducts provided a minimum acceptable tolerance for #9 bars. Figures 5.18a and 5.18b show positions of connectors in a vertical duct and single-line grout pocket connection, respectively.

Caps for the column bent were set without difficulty, prompting the contractor to highly appraise the plate-and-leveling-nut option. Steel shims were stiff and stable. Although caps appeared stable under self-weight, bolts were tightened and timbers were stacked underneath to ensure stability during grouting. Figure 5.18c shows positions of connectors in the double-line grout pocket.

Based on this experience, the contractor made several suggestions (Table 5.2). Especially important were: 1) steel shims rather than plastic shims should be used to provide better stability during cap setting, 2) two shims per column or pile are recommended for better reliability and stability, even if one shim eventually does not bear, 3) ducts in the cap should be no smaller than 3-in. diameter when used with 1-in. bolts, and 4) plates with leveling nuts are not only feasible but recommended, due to ease of construction, adjustability and cap stability.

a. Vertical Duct *b. Single Line Grout Pocket* *c. Bolted Connection*

Figure 5.17 Cap Setting—Trestle Pile Bent



a. Vertical Duct *b. Single-Line Pocket* *c. Double-Line Pocket*

Figure 5.18 Positions of Connectors after Cap Setting

5.2.5 Grouting Operations

To help ensure success in grouting, the precast connection specification requires the contractor to conduct trial batches with candidate grouts prior to actual grouting of the connection (see Chapter 7). Grouting operations included: 1) trial batches, 2) grouting of the column and pile ducts, and 3) grouting of the connections.

5.2.5.1 Trial Batches

Mixing trial batches for candidate grouts enabled the contractor to assess the suitability of a grout for constructability and strength. The specific purposes of a trial batch are to:

- Determine the required amount of water to be added to a particular grout brand to achieve acceptable flowability using the Flow Cone Method per CRD-C 611/ASTM C 939 under the temperature conditions expected in the field
- Determine the grout cube strength corresponding to the efflux time (flow) achieved
- Examine grout for undesirable properties such as segregation
- Establish the adequacy of proposed grouting equipment such as the mixer, tremie tubes, funnels, buckets, and vent tubes
- Provide the contractor with experience in mixing and handling grout prior to connection grouting
- Help the contractor make a judicious decision regarding grout brand.

Trial batches were required in Phase 3, although the three grout brands (MF928, EHF, and Sika 212) had already been selected. The contractor mixed grout in a 5-gallon bucket according to manufacturer's recommendations (Figure 5.19a). Flow was determined using the ASTM Flow Cone Method (Figure 5.19b). A TxDOT Materials representative assisted in conducting the flow cone test and prepared and cured grout cubes.

A slight increase in water was required to achieve a flow of 30 seconds or less. Similar to Phase 2 results, significant differences in strength were obtained for the different grout brands (Table 5.4). Only MF928 achieved strengths in accordance with the manufacturer's specification. Noticeable amounts of air bubbles were observed after mixing EHF. Sika 212 exhibited segregation. No clumps were found in the grouts after mixing with a hand drill. A trial pouring operation was not performed.

a. Grout Mixing *b. Flow Cone Test*

Figure 5.19 Trial Batch

Table 5.4 Grout Flow and Strength for Trial Batch

Grout Brand	Water (lbs/bag)	Temp (F)	Flow (sec)	Grout Cube Strength (ksi)	Notes
EHF	10.2	77	26	2.1, 2.0, 3.3, 5.6 (1-, 3-, 7-, 30-day)	Noticeable air bubbles
Sika 212	8.9	77	24	3.1, 3.5, 3.9. 6.3 (1-, 3-, 7-, 30-day)	Segregation observed
MF928	10.0	77	28	4.1, 4.6, 5.8, 7.4 (1-, 3-, 7-, 30-day)	

The contractor commented that the trial batch was an important step for successful grouting, especially because grouting was a new operation. The contractor also recommended that the steps outlined in the specification be closely followed and that trial batches be completed 1-2 weeks prior to actual grouting operations to ensure that an acceptable grout is selected in time and that the lessons learned from the trial batch are implemented in connection grouting.

5.2.5.2 Connection Grouting

Grouting operations for connections included forming the bedding layer, prewatering and draining, gravity-flow grouting, and curing. Figure 5.20 shows the contractor's approach to forming the bedding layers. Figure 5.20a shows the arrangement of plywood and lumber used to form around the bedding layer for the embedded pile case, whereas 5.20b shows formwork used for the surface-flush case. Figure 5.20c shows the flexible Symons fiberglass forms that were wrapped around the round columns and bolted.

Grouting operations required two days, due primarily to a rain delay. This break, however, provided time for the contractor to evaluate and improve grouting operations. Grouting was conducted using a bucket for some connections and a modified tremie-tube approach for other connections. The following two sections summarize grouting operations. Table 5.5 summarizes grouting data and includes notes on grouting. Lessons learned are summarized in Section 5.2.5.5.

Table 5.5 Phase 3 Connection Grouting Data

Test	Grout Brand	Water (lbs/bag)	Temp (F)	Flow (sec)	Grout Cube Strength (ksi)	Grouting Approach	Notes
PVDE	MF928	10	65	28	2.5, 4.7, 4.2 (1-, 4-, 7-day/ test)	Tremie Tube	Constant drizzle during grouting. Prewatering not conducted. Mortar mixer used for mixing. Continuous tremie tubing not used as funnel capacity was inadequate. Mesh not used. Clumps may have blocked air vent. Vent unclogged with wire. 20 minutes to grout.
PSL	EHF	10	65	45	2.0*, 2.0* (3-, 6-day/ test-1); *PSLE	Bucket	Prewatering not conducted originally. Leaky forms led to prewatering for this and subsequent connections. Holes were drilled for venting during grouting. Grout with 45 sec flow was used as limited numbers of bags were available and connection was more suitable for longer flow (grout pocket). Minor segregation observed. Mesh removed significant clumps. Bubbling of grout evident. 20 minutes to grout. Cube strength corresponds to PSLE cubes.
PSLE	EHF	11	65	19	2.0, 2.0 (3-, 6-day/ test-1)	Bucket	Very similar to PSL grouting. 10% water added to lower flow time based on PSL grouting.
PBC	Sika 212	10	65	17	4.0*, 5.1* (3-, 6-day/ test) *CDL	Modified Tremie Tube	Significant segregation. Leaky forms evident. Continuous tremie tube grouting attempted but flow not well maintained. Nuts tightened prior to grout setting. Cube strength corresponds to CDL cubes.
CVD	MF928	10	65	41, 14, 15	2.5, 4.7, 4.2 (1-, 4-, 7-day/test)	Tremie Tube	Constant drizzle during grouting. Prewatering not conducted. 5-gallon bucket rather than mortar mixer was used for mixing; therefore, grout was not prepared for continuous tremie tubing. Water not measured accurately initially, resulting in 41 sec flow for first bucket. Flow for next two buckets was much lower. 26 minutes to grout.
CDL	Sika 212	10	65	15	4.0, 5.1 (3-, 6-day/test)	Bucket	Significant segregation. Significant layer of water appeared at top of pocket.
CBC	EHF	11	65	45, 18	2.6, 2.9 (3-, 6-day/test)	Modified Tremie Tube	10% water added to lower flow from 45 sec to 18 sec. Retempered mix 1.5 min. Grouting through 2-in. grout port. Continuous flow not feasible with small funnel. Because air vents not used at bedding layer, non-continuous flow enabled air to vent through port. Leaky formwork at bedding layer. Minor segregation. Nuts tightened prior to grout setting.

a. Single Line *b. Single Line Embedded* *c. Fiberglass Collar*

Figure 5.20 Bedding Formwork

5.2.5.3 Tremie-tube Grouting

5.2.5.3.1 CVD and PVDE

Masterflow 928 (MF928) neat grout was used for the CVD and CBC connections. Despite light to moderate rain, it was decided to proceed with grouting operations. Workers appeared slightly hurried. Because the water level based on trial batches was not marked on the bucket, water had to be added to achieve an acceptable flow (Table 5.5). UT and TxDOT representatives made recommendations when appropriate. For example, prewatering was requested to verify tightness of forms after some forms leaked. Grout was initially mixed in 5-gallon buckets, although a much larger volume was required for continuous tremie-tube grouting. The contractor grouted one bucket at a time to avoid waiting 20 minutes for all grout to be prepared (Figures 5.21). This discontinuity in grouting operations caused air to be added into the connection with each new bucket. Fortunately, the low grout viscosity and small air vents (Figure 5.21c) enabled air to adequately surface. Similar to Phase 2 grouting, grout was fed through one duct to fill the bedding and other ducts. Grout was not tamped. Ducts required only minor topping off. Grouting was completed in approximately 30 minutes, the maximum expected working time for MF928. Exposed top surfaces were covered with wet rags, and forms were removed the next day. Despite less than ideal weather and grouting conditions, the connection was grouted adequately, without voids or gaps (Figure 5.22). The success in grouting the first connection is attributed to the excellent performance of MF928, i.e., suitable flow and excellent consistency, without segregation. Table 5.5 shows that the amount of water used in CVD lowered strength significantly from trial batches, although strengths were still close to strengths required by the specification.

a. Lifting Bucket *b. Plan View* *c. Air Vent*

Figure 5.21 Bedding after Tremie-tube Grouting

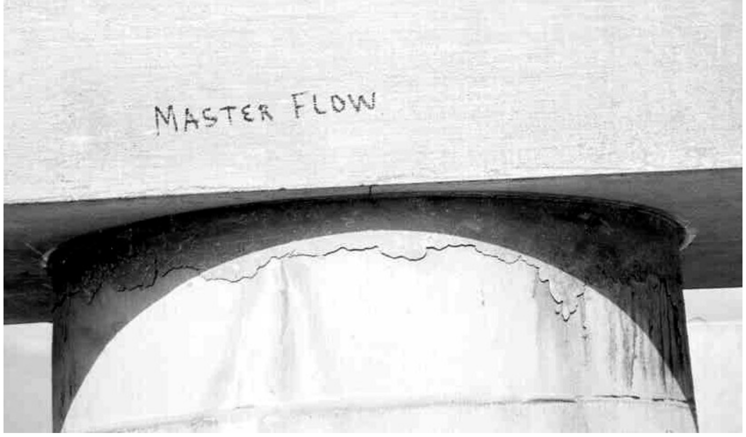
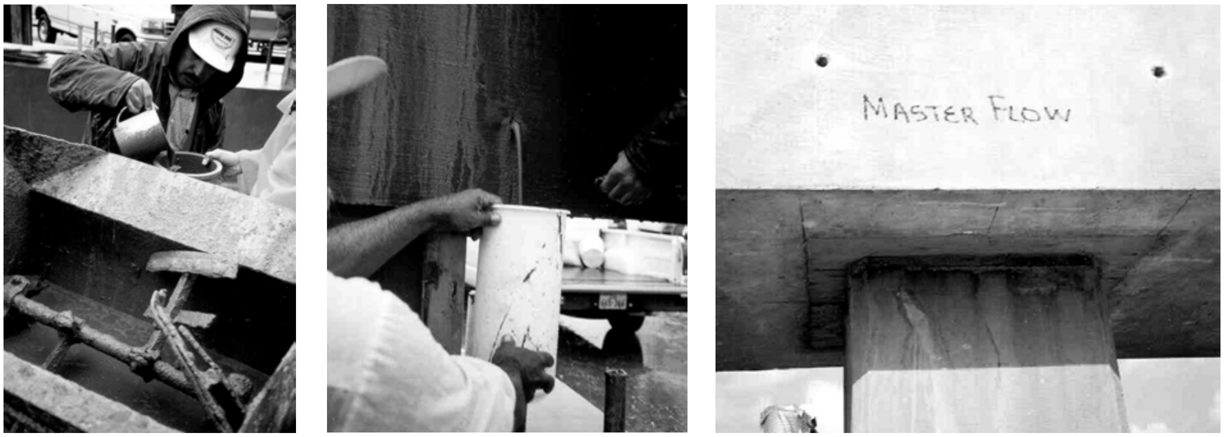


Figure 5.22 Bedding after Tremie-tube Grouting

The research team suggested the contractor use a mortar mixer to mix an adequate grout volume for continuous tremie-tube grouting of PVDE (Figure 5.23a). Continuous flow, however, was not achieved, as the funnel was too small to be kept full during grouting. Figure 5.23b shows the venting of air during grouting. A heavy-gage wire was used to unclog one of the 0.5-in. vent tubes. Blockage was caused by grout clumps, which tend to form due to the lower mixing speed of a mortar mixer compared to a hand drill. A 0.5-in. mesh should be used to filter out grout clumps when pouring grout from the mixer into buckets. Grouting was completed in 20 minutes, two-thirds the time required for the first grouting operation. Figure 5.23c shows the exposed surface of the bedding layer after form removal.



a. Mixer *b. Air Venting* *c. Bedding after Grouting*

Figure 5.23 Tremie-tube Grouting of Single Line Grout Pocket (Embedded)

5.2.5.3.2 PBC and CBC

Sika 212 neat grout was used for the PBC connection (Figure 5.24a). Sika 212 achieved a 17-second flow, but segregation was evident during mixing (Table 5.5). Continuous tremie-tube grouting was not possible due to the limited funnel size. Upon grouting, the forms were found to be leaky and required sealing (Figure 5.24b). Grout in ducts was lightly tamped with a rebar to help remove embedded air. After grouting, plates were placed on a bedding of grout before nuts were installed and tightened snug with a 16-in. spud wrench prior to grout setting. Plates were set on a grout bedding at the cap top. The bedding layer appeared well grouted after form removal (Figure 5.24c).

a. Grouting *b. Plugging Leaks* *c. Bedding after Grouting*

Figure 5.24 Tremie-tube Grouting of Bolted Connection (Pile Bent)

CBC used EHF neat grout. A 2-in. grout port was used in grouting CBC, as shown in Figure 5.25a. No air vents were provided at the bedding layer. However, air was not entrapped because a continuous grout flow was not feasible. Nuts were tightened snug before the grout set. Figure 5.25b shows the cap top anchorage.

a. 2-in. Grout Tube *b. Cap Top Anchorage*

Figure 5.25 Tremie-tube Grouting of Bolted Connection (Column Bent)

5.2.5.4 Bucket Method

5.2.5.4.1 PSL and PSLE

To improve grouting operations the second day, a meeting between the research team and contractor was held to discuss the following:

- Measurement of water based on trial batches
- Grout preparation using a mortar mixer to ensure proper and timely placement of grout
- Larger funnel for tremie-tube grouting

- Larger air vents at the bedding layer
- Prewatering to verify form tightness and prevent loss of moisture
- 0.5-in. mesh to remove grout clumps
- Air removal prior to plugging air vents

After the meeting, the contractor grouted PSL and PSLE using Euclid High Flow (EHF) neat grout. Nearly identical operations were conducted for the two connections. Similar to Phase 2 results, a significant amount of bubbling and clumps were evident after mixing (Figure 5.26), as well as a minor amount of segregation. Figures 5.27a and 5.27b illustrate the simple process of bucket grouting and tamping. After grouting, curing compound was sprayed on the pocket top. The bedding layers were well grouted. Figure 5.27c shows the PSLE pocket surface after hardening. The top surfaces for PSL and PSLE portrayed the same soft porous texture observed in Phase 2. As shown in Table 5.5, the grout strength was only 2.0 ksi at 6 days.

a. Bubbling of Grout *b. Grout Clumps*

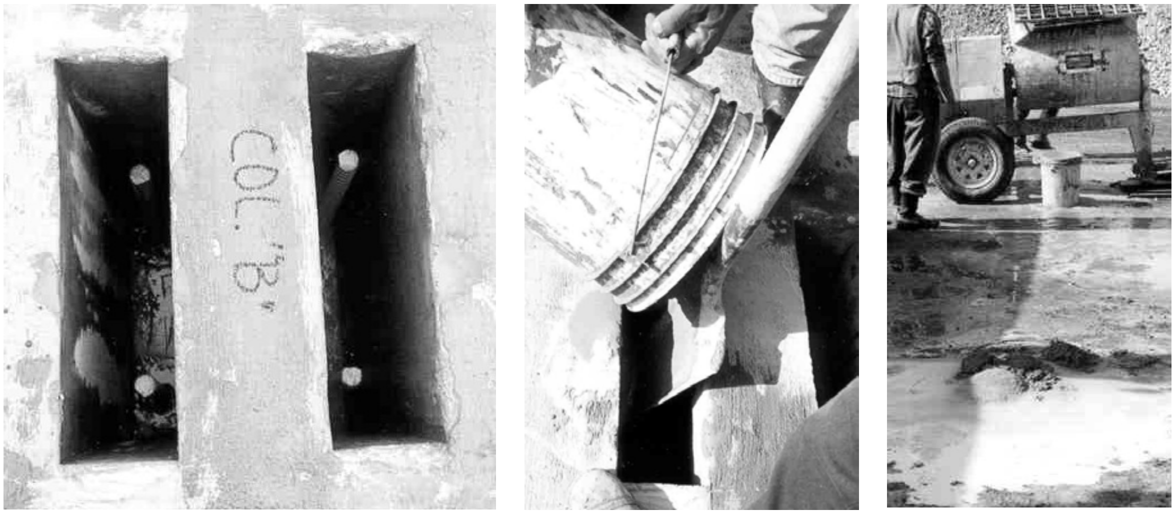
Figure 5.26 Euclid High Flow Grout Used in PSL and PSLE Grouting

a. Placement with Bucket *b. Tamping* *c. Post-grout Surface*

Figure 5.27 Bucket Grouting of Single-Line Grout Pocket

5.2.5.4.1 CDL

CDL was grouted with Sika 212 using a bucket approach as shown in Figures 5.28a and 5.28b. Overall grout characteristics were very similar to the Sika 212 grout used for the Phase 2 CDL connection. A flow of just 15 seconds was obtained, accompanied by significant segregation of the grout (Figure 5.28c). Fluid at the top of the bucket poured out, but aggregate stuck to the bottom of the bucket. While this did not affect grouting procedures appreciably, it is believed that such segregation may have led to water pockets at the bedding layer that resulted in voids after evaporation, as shown in Figures 5.29a and 5.29b. These voids were very similar in size to those that developed during Phase 2 grouting with Sika 212. Evaporation of water and paste also formed a 2-in. deep cavity at the top of the pockets (Figure 5.29c). Gaps and cavities can pose serious durability concerns.



a. Connectors *b. Bucketing* *c. Segregation*

Figure 5.28 Bucket Grouting of Double-Line Grout Pocket

a. Bedding Gap *b. Close-up of Gap* *c. Cavities*

Figure 5.29 Gaps and Cavities at Double-Line Grout Pocket Connection

5.2.5.5 Lessons Learned

Numerous lessons were learned through each stage of construction. The following sections summarize the primary lessons. Many of these lessons are closely related to the contractor's comments summarized in Table 5.2.

5.2.5.5.1 Fabrication

1. Fabricating caps with grout pockets and corrugated ducts is feasible and relatively simple. Grout pockets may be simpler to fabricate due to larger available tolerances that simplify form placement.
2. Wood forms for grout pockets are preferable to Styrofoam for handling and placement. Mass production favors removable steel forms.
3. Placement of spiral confining reinforcement is simple due to compressibility of the spirals. Placement of welded wire fabric is expected to be difficult and is not recommended.
4. The diameter for ducts embedded in piles and columns should be maximized to provide sufficient tolerances for duct and connector placement. Ducts should provide a tolerance of at least +/-0.75 in. for connectors, although a 1-in. minimum is preferable. Potential conflict with pile or column reinforcement should be considered and avoided.
5. Accurate positioning of ducts and connectors may be achieved using templates and/or supplementary reinforcement. Templates should include a central opening to facilitate casting and may be fabricated with the ducts in a one-piece assembly. Care is required to prevent movement of supplementary reinforcement.

5.2.5.5.2 Cap Setting

1. Cap setting for multiple connections presented no difficulties. Horizontal tolerances should be maximized wherever possible.
2. Steel shims provide better stability than plastic shims during cap setting. Two shims per column should be used to provide more reliable bearing.
3. Use of plates and leveling nuts is a feasible alternative for cap support. Provisions for cap adjustability and stability are important advantages.
4. The diameter for ducts embedded in the cap should be maximized to provide sufficient tolerances for connector placement. Tolerances should be at least +/-1 in., although +/-1.5 in. is preferable.

5.2.5.5.3 Trial Batch

1. A trial batch for candidate grouts is essential to verify grout properties and compare grout brands, as well as to provide experience for the contractor. This is an essential step for proper grout selection.
2. A trial batch should be conducted sufficiently early to ensure lessons learned are implemented in actual grouting operations.
3. A trial batch should include use of proposed grouting procedures and equipment.
4. Accurate determination of grout flow requires that the flow cone test use a representative portion of the grout.

5.2.5.5.4 Grouting

1. Grouting procedures should be listed in plan sheets to ensure procedures are accurately implemented.
2. An on-site pre-grouting meeting between the contractor and a TxDOT representative should be conducted just prior to actual grouting operations to review the grouting procedure and ensure lessons learned during the trial batch process are incorporated.

3. The TxDOT representative should be available for consultation during initial grouting operations and periodically thereafter. Because of the difficulty in correcting field problems after grouting, special care and oversight should be exercised prior to and during initial grouting operations.
4. Grouting should be limited to the same weather restrictions as for casting concrete.
5. Prewatering is necessary to ensure forms are tight and to prevent moisture loss to the concrete from the grout.
6. A 0.5-in. wire mesh should be used to remove clumps from the grout after mixing.
7. Gravity-flow grouting is feasible for all connection types. Grout pockets may use either the bucket method or tremie-tube grouting. Connections with ducts should use tremie-tube grouting to prevent air entrapment.
8. To ensure continuous flow during tremie-tube grouting, grout should be mixed in a sufficient quantity and poured using a funnel of sufficient volume capacity.
9. Vent tubes are required to vent air at the bedding layer. Use of a fluid grout with a flow of no more than 30 seconds facilitated surfacing of air during grouting.
10. The use of shims poses no difficulties in grouting the bedding layer.
11. Tamping provides an adequate means to consolidate grouted connections.
12. Grouting may require an average of approximately 15-20 minutes per connection. However, grouting of initial connections may require a longer time. This should be considered in grout selection.
13. Exposed grout surfaces should be carefully examined for voids after grout setting and form removal.
14. Grouts exhibiting segregation during trial batches should not be used for connection grouting, due to potential formation of gaps at the bedding layer and cavities at the cap top, which may threaten durability as well as force transfer.
15. Connection configurations that embed piles or columns into the cap provide a more limited path for moisture ingress and thus may enhance durability. However, the bedding layer cannot be inspected after grouting in such cases.
16. Surface sealants may be applied to exposed grout surfaces at the cap top or bedding layer to enhance connection durability.

5.2.6 Materials

5.2.6.1 Steel

See Section 4.2.5.1 and Table 4.4 of Chapter 4 for material properties of reinforcing steel, connectors, and ducts.

5.2.6.2 Concrete

A TxDOT Class C mix with a 4-in. to 6-in. slump and a minimum 28-day compressive strength of 3600 psi was used. See Section 3.2.4.3 for further details. Compressive strength curves are plotted in Figure 5.30 and show a 28-day strength of approximately 5700 psi and 7000 psi for the pile and column bents, respectively. The 28-day strength for column and pile stubs was approximately 6400 psi. Approximate test-day strengths for the column bent were 5.9 ksi for the cap and footing base and 6.3 ksi for the column stub. For the pile bent, approximate test day strengths were 7.2 ksi for the cap and footing base and 6.3 ksi for the pile stub.

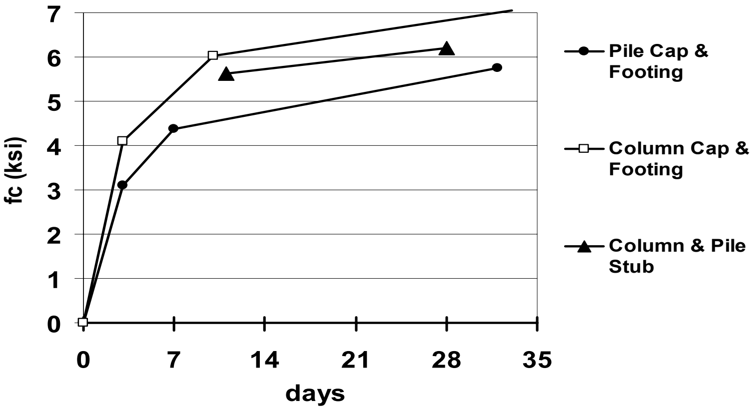


Figure 5.30 Concrete Compressive Strength Curves

5.2.6.3 Grout

EHF, Sika 212, and MF928 neat grouts were used for trial batches, grouting connectors into the piles and columns, as well as connection grouting. Figure 5.31 compares grout strengths for trial batches, and Figure 5.32 plots grout strengths for connector grouting into piles and columns and connection grouting.

5.2.7 Test Setup and Instrumentation

As shown in Figures 5.1b and 5.2b, the test setup was designed for vertical loading of the pile bent and column bent. The test setup consisted of a series of 100-ton rams positioned on both sides of connections, with W14x398 girders that transferred reactions into the footing using 1.25-in. threaded rods. Tie rods were anchored with anchor plates and detailing shown in Figures 5.3 and 5.4. Rams were placed on the cap top at a 9-in. longitudinal eccentricity for most tests. The use of multiple pumps enabled application of loads corresponding to transverse and longitudinal moments. Figures 5.5 and 5.6 show the configuration of rams and load combinations used for the pile and column bents, respectively.

Field instrumentation was limited to displacement dial gages placed at the underside of the bent cap, as shown in Figure 5.33. For the pile bent, dial gages were placed at centerline of the beam (+/-1 in.) at ram positions along the bent for the transverse moment test, but were moved 1 in. from the edge for other tests (Figure 5.34). For the column bent, dial gages remained at the centerline of the beam for all tests (Figure 5.35). The applied load was measured using pressure gages attached to the pump manifold. The specimen was inspected for development and extension of cracks at each load increment. Cracks were typically measured at 20- to 40-kip intervals during loading, and crack patterns were marked on the specimens.

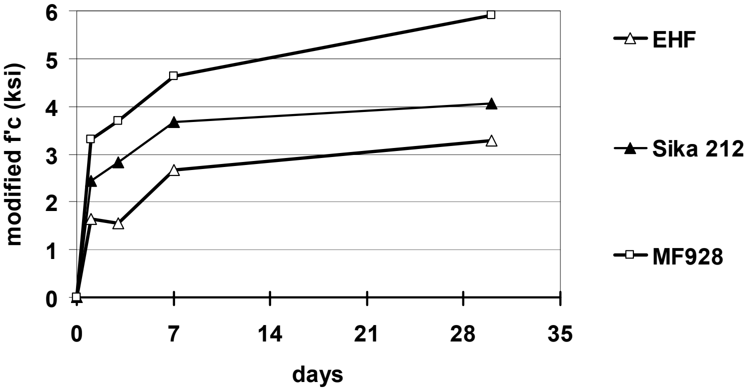


Figure 5.31 Trial Batch Grout Compressive Strength Curves

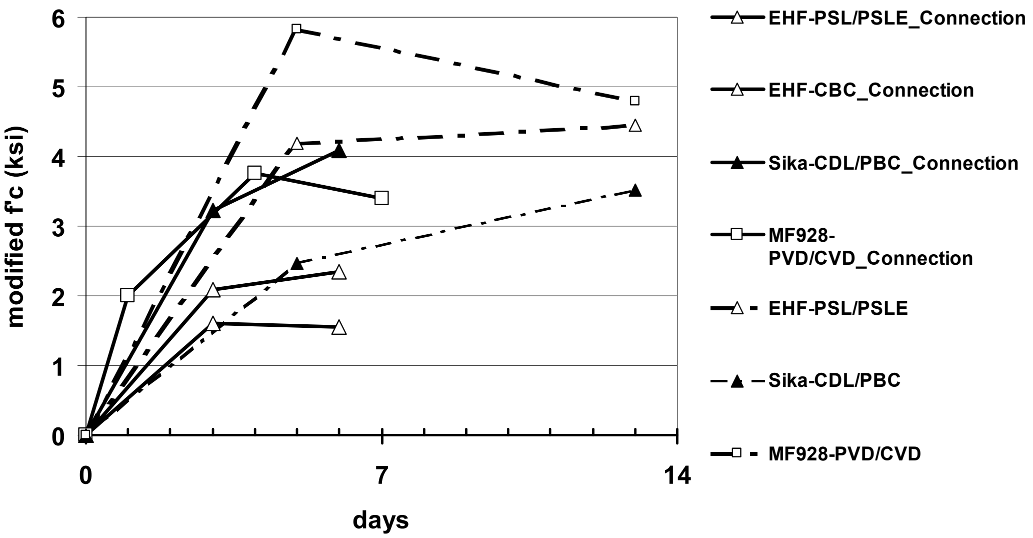
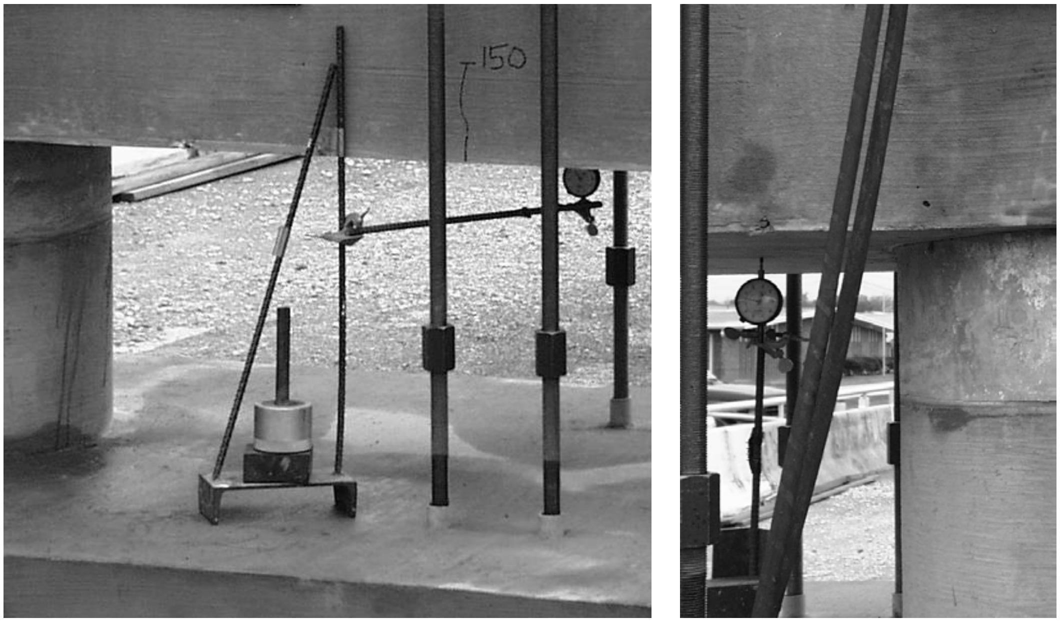


Figure 5.32 Connection Grout Compressive Strength Curves



a. Dial Gage and Stand *b. Dial Gage*

Figure 5.33 Dial Gage

5.2.8 Test Procedure

Testing was conducted by loading the vertical rams with pneumatically-driven hydraulic pumps (Figure 5.1b). Figures 5.34 and 5.35 show the configuration of rams, level of load, and eccentricities used in the test series for each bent. Load was applied at a rate of approximately 40 kips per minute. Load was held constant at each load increment while the specimen was inspected and crack growth marked. The average load for each pair of rams was marked on the specimen.

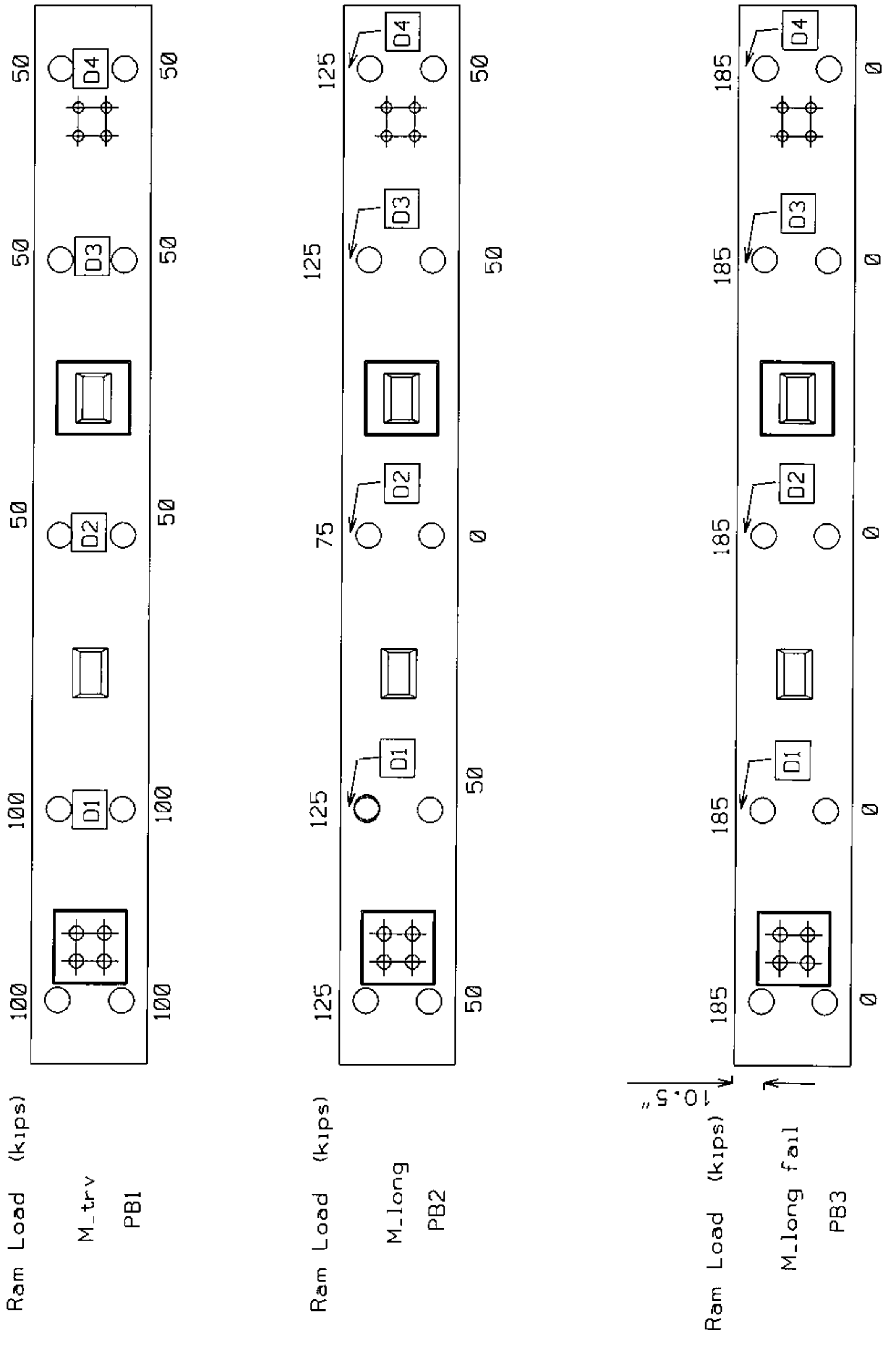


Figure 5.34 Dial Gage Locations—Pile Bent

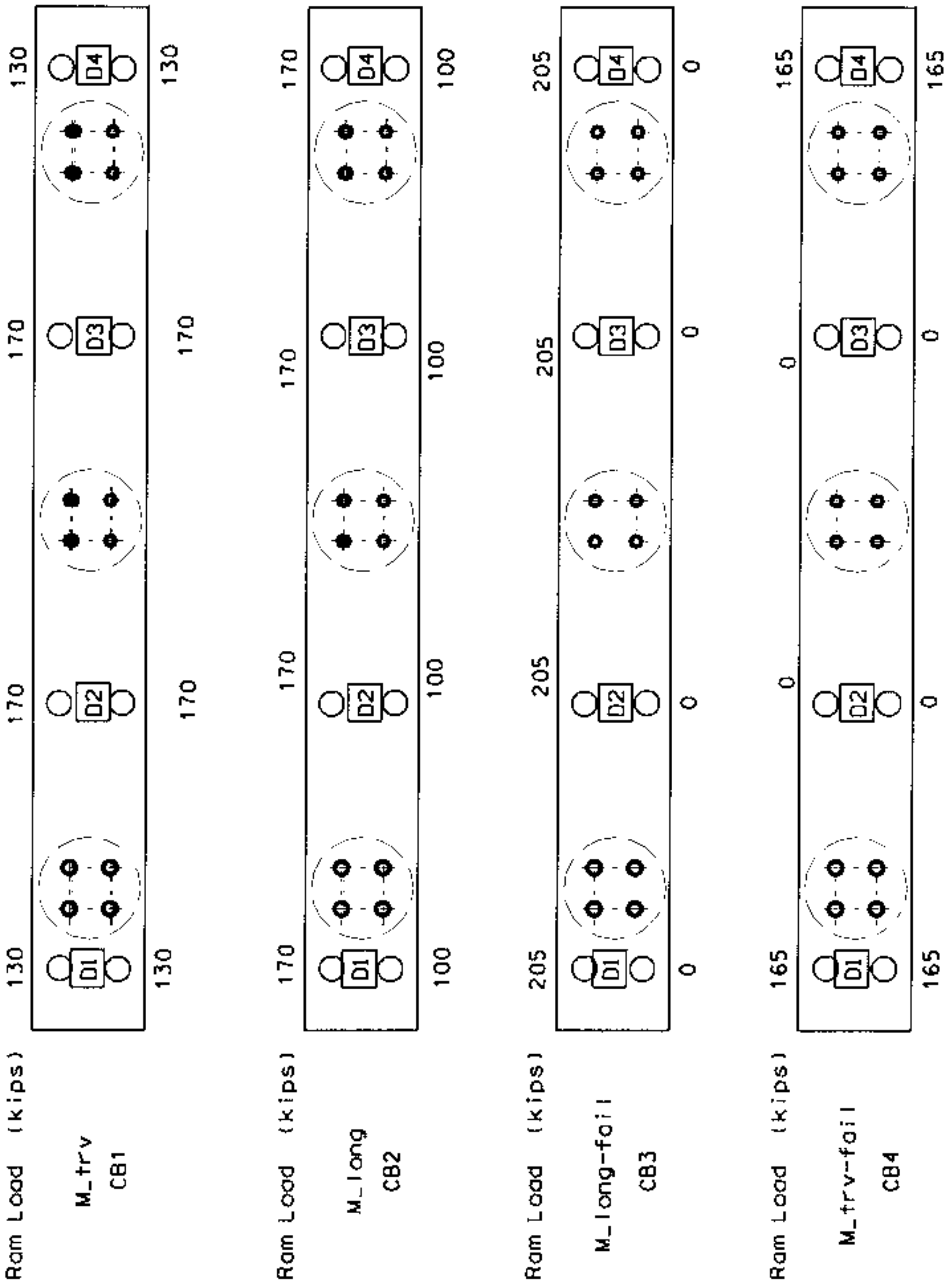


Figure 5.35 Dial Gage Locations—Column Bent

5.3 TRESTLE PILE BENT

5.3.1 Summary of Testing

The secondary objective of Phase 3 was to examine connection and bent behavior at service- and factored-level vertical loads, as well as to observe failure modes for the trestle pile bent. Phase 3 testing included three pile bent tests, PB1-PB3, which investigated connection behavior under loads corresponding to the transverse and/or longitudinal eccentricities shown in Figure 5.5.

Table 5.6 shows the test matrix and primary results, including P_{max}/P_n ratios and maximum deflections along the bent. P_{max}/P_n is the ratio of the maximum applied load to an equivalent axial load capacity at the top of the pile or column, based on the Reciprocal Load Method. As expected from the low P_{max}/P_n ratios, negligible distress developed in the connection region during the first two proof tests. In fact, no cracks in the connection region or along the cap developed. Deflections were minor and corresponded closely to analytical predictions assuming rigid connections. No discernable difference in response was evident between surface-flush (PSL) and embedded-pile (PSLE) connections. The use of plastic shims did not affect connection response.

PB3 loaded the bent using a 10.5 in. longitudinal eccentricity to produce failure. As suggested by the P_{max}/P_n ratios greater than 1.0, bar yield developed, followed by crushing at the column tops. Strength was predicted reasonably well and specimens exhibited excellent ductility. Pile crack widths as large as 0.02 in. and openings of more than 0.1 in. at the bedding layer developed, although the bent cap itself did not crack. Load-deflection plots exhibited highly nonlinear behavior. There was no evidence of significant slip at grout-concrete interfaces and bedding layers performed well.

Table 5.6 Pile Bent Test Results

Test ID	Pmax/Pn[A]				Deflection[B] (in.*0.001)				Notes
	PVDE	PSL	PSLE	PBC	D1	D2	D3	D4	
PB1—Mtrv	0.23	0.11	0.07	0.11	8	4	5	6	No cracks on cap or pile
PB2—Mlong	0.31	0.29	0.29	0.31	28	26	31	32	No cracks on cap or pile
PB3—Mlong-fail	1.20	1.05	1.05	1.20	149	181	186	180	Bar yield; column crushing

A. Pmax/Pn is the ratio of maximum applied load to an equivalent axial load capacity at the column top, based on the Reciprocal Load Method.

B. Listed deflection is maximum deflection at centerline of the cap for Test 1 and maximum deflection 1 in. from edge for Tests 2 and 3.

5.3.2 PB1

As shown in Figure 5.5, the PB1 test loaded the specimen up to the factored level with a transverse eccentricity. Based on a 2-D frame analysis, the maximum transverse eccentricity was 0.4 in. or less at column tops. Loading produced negligible distress in the connection region and bent cap. Figure 5.36 shows the axial load-uniaxial moment strength interaction curves for each connection and data points for connections with the governing P_{max}/P_n ratio. Table 5.6 shows P_{max}/P_n ratios no larger than 0.23 (PVDE). No cracks in connection regions or along the cap developed during testing.

Figure 5.37 shows the PB1 load-deflection response. Loading produced very minor deflections, less than 0.01 in. Measurements at all gage locations were within approximately 10% of predicted response based on the assumption of rigid connections. Thus, negligible difference in response for the PSL and PSLE connections resulted.

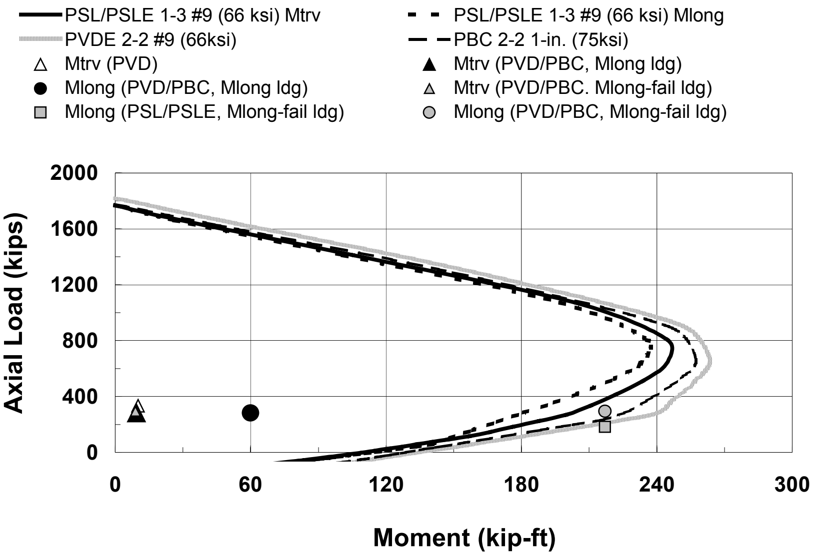


Figure 5.36 Axial Load-Moment Interaction Diagram for Pile Bent Tests

5.3.3 PB2

Figure 5.34 shows the loading configuration for PB2, with rams positioned for a maximum realistic longitudinal moment and the corresponding transverse moment. Transverse and longitudinal eccentricity combinations, respectively, were 0.4 in. and 2.5 in. for exterior columns, and 0.6 in. and 6.1 in. for interior connections. Again, no cracks developed in the connection region and bent cap. Minor levels of response were expected (Figure 5.36 and Table 5.6).

Figure 5.37 shows the PB2 load-deflection response. Loading produced a maximum vertical deflection of only approximately 0.03 in. under the rams. Rigid rotation of the cap was evident by the nearly equal deflections along the cap edge. However, no opening at the bedding layer was detected. In contrast to PB1, PSL and PSLE connections were loaded identically. Comparisons between deflections D1-D2 and D2-D3 show negligible difference in PSL and PSLE connection response. However, this may be attributed primarily to rotation of the torsionally stiff cap.

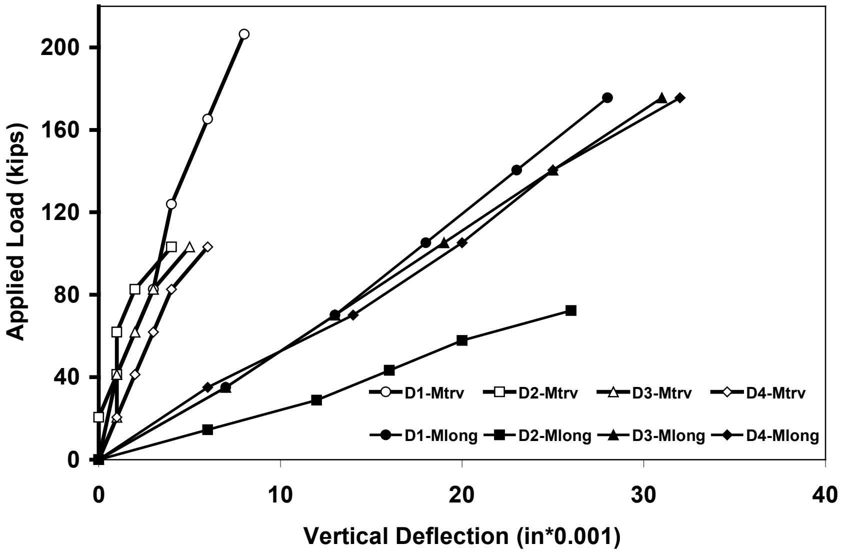


Figure 5.37 Load-Deflection Behavior for PB1 (Mtrv) and PB2 (Mlong)

5.3.4 PB3

PB3 loaded the bent with a line of rams at a longitudinal eccentricity of 10.5 in. on one edge (Figure 5.34). The maximum eccentricity combination at a column top was 0.6 in. in the transverse direction and 6.1 in. in the longitudinal direction, for interior connections. Under load, the cap rotated and the bedding layer gradually opened on the tension side, leading to bar yield and crushing at the pile top. The failure load was reasonably well predicted, as shown by Figure 5.36 and P_{max}/P_n ratios in Table 5.6. Excellent ductility was exhibited.

Figure 5.38 shows the nonlinear load-deflection behavior. Nonlinear behavior corresponded to the development and extension of cracks in the pile and bedding layer. Figures 5.39 and 5.40 show crack patterns on each pile, which were similar to those that developed in the failure tests of the Phase 2 PSL connection. Cracks formed at 20 to 40 kips, but crack widths did not exceed 0.013 in. until a load of 80 kips or larger. Based on load-deflection response, bar yield likely occurred at the load step between 120 and 160 kips. This matches the expected response based on the axial load-moment strength interaction curve. The bedding layer opened to approximately 0.03 in. at 120 kips.

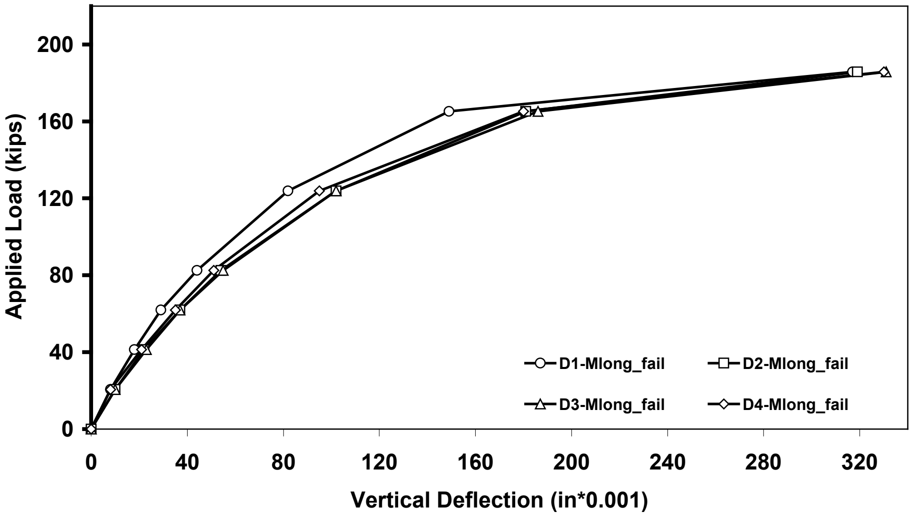
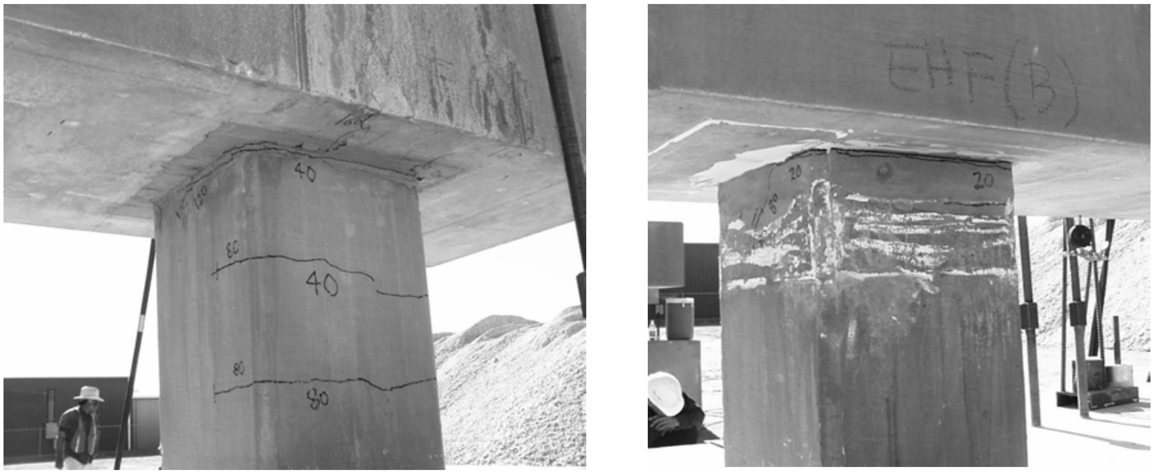


Figure 5.38 Load-Deflection Behavior for PB3 (Mlong-fail)



a. PVDE *b. PSL*

Figure 5.39 PB3 Pile Cracks—PVDE and PSL

a. PSLE *b. PBC*

Figure 5.40 PB3 Pile Cracks—PSLE and PBC

At a load of 160 kips, crushing appeared initially at PVDE, but shortly thereafter crushing developed at the top of all piles due to the rigid rotation of the cap. This was accompanied by a large opening on the tension side of the bedding layer. Figures 5.41 and 5.42 show the crushing and spalling of concrete at failure. No connection distress was evident in the connection region above the bedding layer or along the bent cap. There was also no evidence of loss of anchorage in piles or the cap, or significant slip at grout-concrete interfaces, and bedding layers performed well. Load deflection plots reveal very similar response for PSL compared to PSLE related to the torsionally stiff cap.

Figure 5.41 Crushing and Spalling at Pile Tops in PB3—Side View

Figure 5.42 Crushing and Spalling at Pile Tops in PB3—Front View

5.4 CAST-IN-PLACE COLUMN BENT

5.4.1 Summary of Testing

Four tests, CB1 to CB4, were conducted on a cast-in-place column bent as shown in the loading configuration in Figure 5.35. Due to project limitations, the cast-in-place bent used close column spacing of 9 ft., resulting in small transverse eccentricities. CB1 and CB2 were proof tests, whereas CB3 and CB4 were "failure" tests using larger eccentricities. Table 5.7 shows the test matrix and primary results, including P_{max}/P_n ratios and maximum deflections along the bent. The very low P_{max}/P_n ratios were indicative of the minor distress that developed in the connection region during all tests. Segregation of the CDL grout resulted in a void at the interior CDL connection. However, the loading configurations produced little moment transfer at this connection.

Table 5.7 Column Bent Test Result

Test ID	**Pmax/Pn**[A]			**Deflection**[B] **(in.*0.001)**			**Notes**
	CVD	**CDL**	**CBC**	**D2**	**D3**	**D4**	
CB1—Mtrv	0.16	0.00	0.16	31	23	15	Flexural cracks on cap and column
CB2—Mlong	0.20	0.10	0.20	21	15	12	Crack growth; bedding layer opening
CB3—Mlong-fail	0.25	0.25	0.25	11	6	8	Bedding layer gaps up to 0.013 in.
CB4—Mtrv-fail	0.35	0.00	0.35	-5	-10	28	Flexural and shear cracks

A. Pmax/Pn is the ratio of maximum applied load to an equivalent axial load capacity at the column top, based on the Reciprocal Load Method.

B. Maximum deflection at centerline of the cap for each test, including column deformations.

Although flexural and shear cracks as wide as 0.007 in. developed along the cap and at the cap top in the connection region, connectors were not significantly challenged. Hairline cracks no larger than 0.013 in. formed at the bedding layer during the longitudinal failure test. Limitations in the test setup and specimen configuration prevented application of loads and eccentricity combinations severe enough to produce failure in the connection region. Deflections at the centerline of the cap were minor and were within approximately 30% of analytical calculations that assumed rigid connections. Load-deflection plots exhibited softening related to flexural cracking along the cap. There was no evidence of significant slip at grout-concrete interfaces. The use of steel shims or plates and leveling nuts did not affect connection response.

5.4.2 CB1

CB1, the first of two proof tests, loaded the specimen to the factored level using a transverse eccentricity (Figure 5.35). Figure 5.43 shows the axial load-uniaxial moment strength interaction curves and data points for connections with the governing P_{max}/P_n ratio. Table 5.7 shows a maximum P_{max}/P_n ratio of only 0.16 at exterior connections. The transverse eccentricity was 1.6 in. at exterior connections. Figure 5.44 shows flexural cracks that developed on the cap top in the negative moment region of CVD, as well as flexural cracks at midspan. Cracks developed at the expected cracking moment and were no larger than 0.007 in. on the cap top at the factored load. Cracks did not appear at the grout surface of ducts. Figure 5.45 shows the CB1 load-deflection response, which exhibits some softening due to flexural cracking. Deflections at the centerline of the cap were minor and, on average, were within approximately 30% of analytical predictions assuming rigid connections.

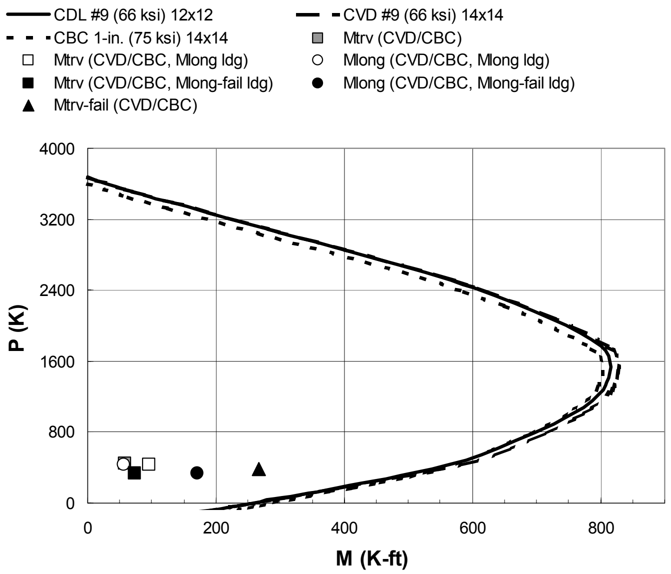


Figure 5.43 Axial Load-Moment Interaction Diagram for Column Bent Tests

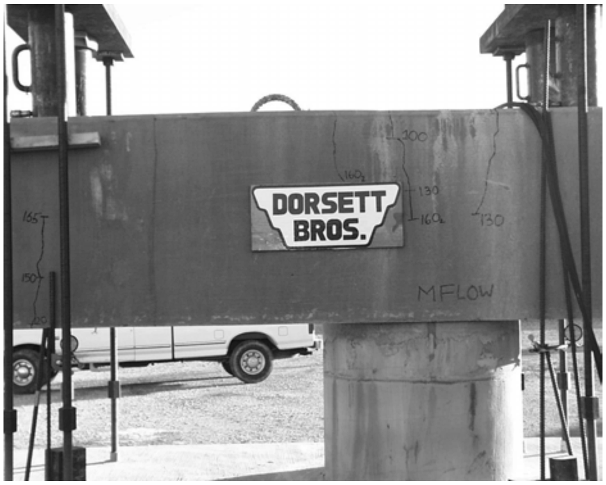


Figure 5.44 Flexural Cracks for CB1

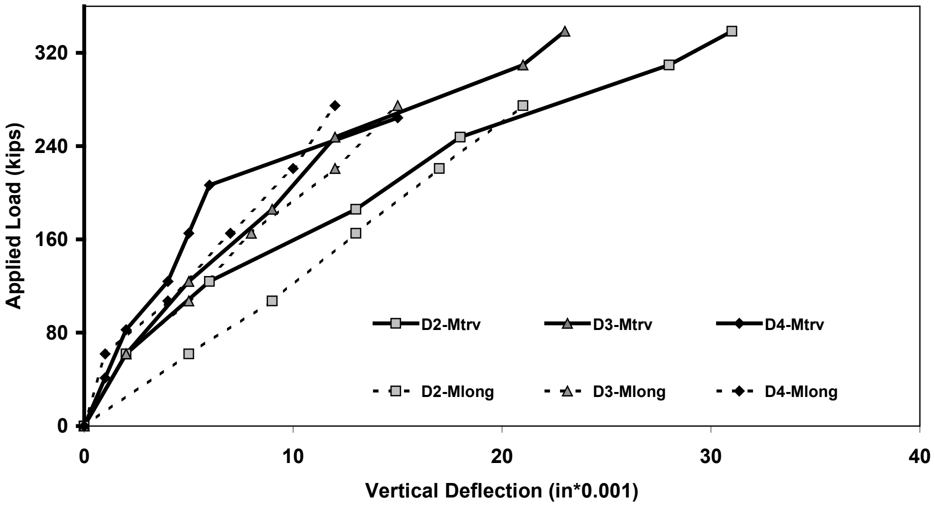


Figure 5.45 Load-Deflection Behavior for CB1 (Mtrv) and CB2 (Mlong)

5.4.3 CB2

CB2 applied a maximum realistic longitudinal moment. Transverse and longitudinal eccentricity combinations, respectively, were 2.7 in. and 1.6 in. for exterior columns. Figure 5.46 shows the slight crack extensions that developed. Slight openings were evident at the bedding layers, particularly at the CDL connection where inadequate connection grouting existed. However, openings were limited to only 0.003 in. Figure 5.47 shows a plan view of cracks that developed in the connection regions at the factored load. Crack widths were limited to approximately 0.007 in.

Figure 5.46 Flexural Cracks for CB2

a. Vertical Duct *b. Double Line* *c. Bolted Connection*

Figure 5.47 Flexural Cracks in Connection Region for CB2

5.4.4 CB3

CB3 attempted to generate a more severe moment transfer to challenge connectors by loading the bent with a line of rams at a longitudinal eccentricity of 10.5 in. on one edge (Figure 5.35). Transverse and longitudinal eccentricity combinations at the column tops were 2.7 in. and 6.2 in., respectively. Under load, the cap rotated and the bedding layers gradually opened on the tension side as large as 0.013 in. Figure 5.48 shows a crack at the CVD bedding layer. Although the bedding layer opened slightly, connectors were not significantly challenged. Deflections measured at the centerline were minor (Figure 5.49). There was also no evidence of anchorage loss in columns or the cap, or significant slip at grout-concrete interfaces.

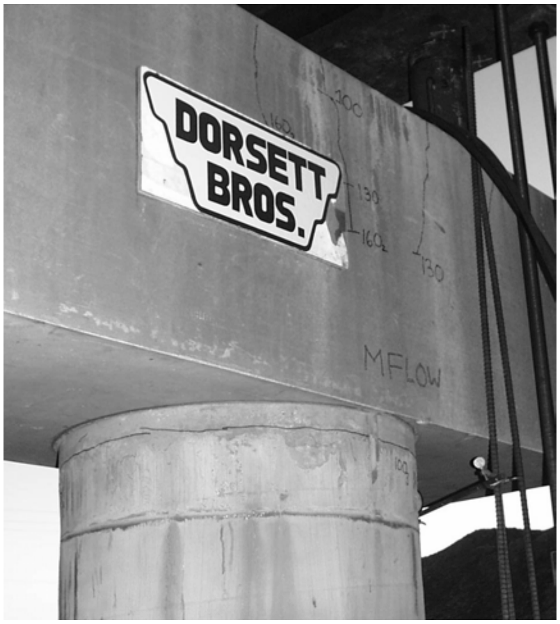


Figure 5.48 Cracks in the connection region of CVD for CB3

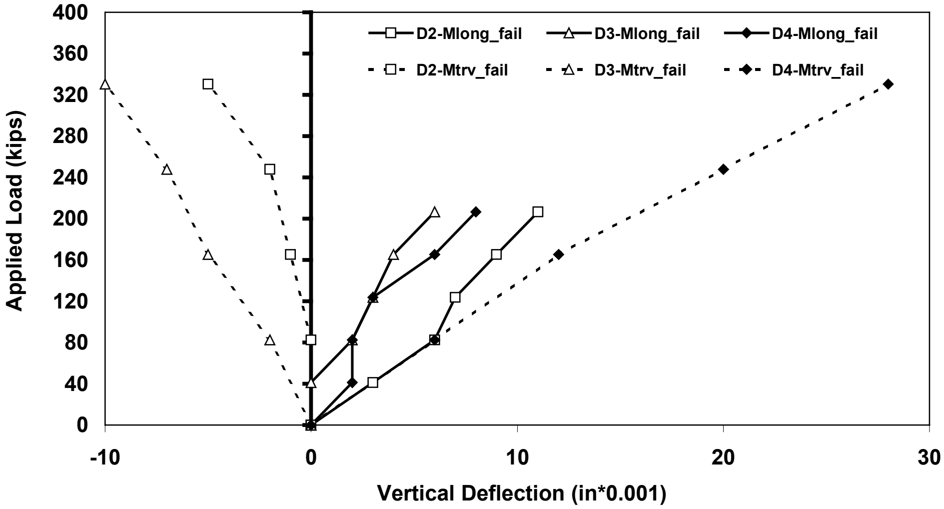


Figure 5.49 Load-Deflection Behavior for CB3 (Mlong-fail) and CB4 (Mtrv-fail)

5.4.5 CB4

CB4 was a final attempt to challenge connectors by applying a large transverse moment by positioning rams at the end spans as shown in Figure 5.35. The transverse eccentricity was 8.3 in. at the column tops of the exterior connections. Additional flexural and shear cracks developed in the cap, but there was no evidence of loss of anchorage in columns or cap, or significant slip at grout-concrete interfaces. The pattern of deflections shown in Figure 5.49 was within approximately 30% of analytical predictions assuming rigid connections.

5.5 CONCLUSIONS

5.5.1 Constructability

Phase 3 construction and testing of full-scale trestle pile and column bents provided a suitable means to assess the ease of construction for single-line and double-line grout pockets, grouted vertical ducts, and bolted connections. All connection types were found to be viable for construction. However, many lessons in fabrication, cap setting, and grouting were learned. These lessons are summarized in Section 5.2.5.5.

5.5.2 Force Transfer

Phase 3 testing examined connection and bent behavior at service- and factored-level vertical loads. Failure was also investigated for the pile bent. The following conclusions can be made:

1. Connection types—*Single-line grout pockets, double-line grout pockets, grouted vertical ducts, and bolted connections are viable alternatives for a precast bent cap system.* Using a minimum number of straight, epoxy-coated #9 bars at a $13d_b$ embedment depth or 1-in. bolts anchored at the cap top, all connection types exhibited the expected level of response at both service and factored loads. Minor distress developed in the connection region and bedding layer. There was also no evidence of loss of anchorage in the cap or columns and piles, or significant slip at grout-concrete interfaces. The pile bent exhibited the expected strength and excellent ductility in the failure test.

2. General behavior for proof tests—*Minor levels of connection distress are expected for pile and column bents under service and factored vertical loads at realistic eccentricities.*

   a. Proof tests for both bents produced minor connection distress at factored vertical loads. Bar pullout of connectors was not significantly challenged. Flexural cracks in the connection region, along the cap, or in the pile did not form. Cracks were limited to 0.007 in. at the cap top for the column bent. The minor deflections that developed for bents corresponded well to analytical predictions based on the assumption of a rigid connection. Rigid rotation of the cap was evident for longitudinal loading, but no opening at the bedding layer developed.

   b. Transverse eccentricities at pile and column tops were considerably smaller than those imposed during the isolated beam-column tests of Phase 2. Though this was partly due to the absence of a horizontal load, it was more a reflection of the range of eccentricities that may be reasonably expected for common trestle pile bent configurations. Small eccentricities for the cast-in-place column bent were due to the use of a rectangular cap with small span lengths (related to testing limitations). Thus, more extensive conclusions can be made for trestle pile bents.

3. Failure mode for trestle pile bent—*Anchorage sufficient to develop the bar yield strength is expected at an embedment depth of $13d_b$ for epoxy-coated bars. In addition, adequate interlock between connectors, grout, ducts, and the concrete cap is expected for all connection types. Strength can be predicted by conventional means. Excellent ductility is expected.* Application of

an unrealistically large eccentricity was required to produce bar yield in connectors. Load-deflection behavior and flexural crack patterns in the cap and bedding layer region indicate excellent ductility. Based on an embedment depth of 13d_b (concrete compressive strength of 7.2 ksi; modified grout cube strength between 2.0 ksi and 4.2 ksi) anchorage was maintained without evidence of splitting cracks in the cap or pile. Under load, the cap rotated, leading to opening of the bedding layers and bar yield on the tension side, followed by crushing at the pile tops on the compression side. The failure load was reasonably predicted by the P_{max}/P_n ratios. No connection distress was evident in the connection region above the bedding layer or along the bent cap. Slip between connectors, grout, ducts, and the concrete cap was minor for all connection types (based on load-deflection response, as well as a lack of splitting cracks in the anchorage region).

4. Confining reinforcement—*Spiral confining reinforcement is not expected to contribute to ductility and strength for connector spacing of 2.5d_b or more and duct spacing of one duct diameter or more. A minimum amount of spiral reinforcement in the connection region of the bent cap is conservatively recommended for use*. Anchorage was most severely challenged in pile failure tests. Connector spacing in the cap for grout pocket connections was 2.5d_b, and duct spacing for the grouted vertical duct connection was 1.0 duct diameter and for the bolted connection 1.7 duct diameters. No distress was observed in the confined area for all connections; in fact, the cap did not exhibit any cracks. Thus, for such connector and duct spacing ductility and strength contributions of confining reinforcement are expected to be negligible. However, closer spacings for ducts and connectors are possible. In addition, because spiral reinforcement requires minimal fabrication and construction effort, it is recommended that a minimum amount of spiral confinement be included in the joint region as a conservative and inexpensive measure. See Chapter 6 for further discussion.

5. Shims—*Steel or plastic shims, as well as plates and leveling nuts, are a viable means to support a precast bent cap on piles or columns and are not expected to affect connection response*. The use of steel or plastic shims or plates and leveling nuts did not produce any discernable effect on connection response in proof tests or failure tests. Construction issues are discussed in Section 5.2.5.5.

6. Bedding layer—*The existence of a bedding layer is not expected to affect connection response. Connections that use a pile or column embedded into the cap are expected to exhibit response similar to connections using a surface-flush pile or column*. Unlike Phase 2 testing, vertical cracks did not form at the exposed portion of the bedding layer under factored loads, even for the 4-in. thick bedding layer used for the bolted connection (CBC). In addition, no cracks appeared at the bottom grout surface at the cap-to-pile interface for the embedded pile specimens during proof tests. Similar pile bent response occurred for the embedded and surface-flush connection options. For large longitudinal eccentricities, similar response is attributed to rotation of cap. An embedded connection may enhance durability because of the more limited path for moisture ingress. Such connections prevent inspection after grouting operations.

7. Grout type and strength—*Masterflow 928, Sika 212, and Euclid Hi-Flow neat grouts provided acceptable force transfer characteristics for application to precast bent cap connections. The use of grout with a lower strength than the surrounding concrete did not adversely affect connection performance even for pile failure tests. However, it is recommended that the grout strength be specified to exceed the concrete strength by a minimum 1000-psi margin*. Despite low strength of EHF grout (4.2 ksi or less on test day) and segregation of Sika 212 grout, adequate transfer of forces developed between connectors, grout, ducts, and concrete for all connections, even for the pile bent failure test. A void at the bedding of the CDL connection did not produce adverse response, although CDL was the least challenged connection during column bent testing. Voids can be a serious concern for durability and force transfer. As mentioned in Chapter 7, it is

recommended that the modified grout cube strength exceed the concrete compressive strength by 1000 psi.

8. Analytical predictions—*Traditional approaches that account for combined flexure and axial force effects as well as biaxial bending can be used to determine connection strength. Prediction of deflections should be based on either a reasonably accurate determination of connection stiffness (using actual connector configurations) or conservative assumptions*. Failure loads were predicted within approximately 20% for the pile bent failure test using axial load-moment (P-M) strength interaction diagrams and the Reciprocal Load Method. Deflections at the centerline of the bent due to transverse eccentricity were within approximately 30% of predictions based on the assumption that connections were rigid. However, design should be based on either a reasonable estimate of the actual connection stiffness or conservative assumptions (such as pins at column and pile tops). This topic is addressed further in Chapter 6.

CHAPTER 6:
DEVELOPMENT OF A DESIGN METHODOLOGY

6.1 INTRODUCTION

This chapter summarizes the development of a design methodology for a precast bent cap system. First, the current approach used by TxDOT to design reinforced concrete interior bents is summarized. Then, a design methodology for a precast bent cap system is introduced. A flow chart is used to illustrate the eight-step procedure. Finally, the development of equations for anchorage of straight or headed connectors in grout pockets or grouted vertical ducts is presented. The appendix provides plan sheets for the first bridge designed using the design recommendations.

6.2 CURRENT DESIGN APPROACH FOR CAST-IN-PLACE BENTS

6.2.1 TxDOT Standard Practice

Over the past several decades, TxDOT has developed a standard practice for design of interior bents, which are usually either multi-column or trestle pile bents. Cast-in-place construction is used for the bent caps, which are supported by either cast-in-place columns or precast piles. Caps are normally rectangular in cross section, although inverted-tee caps have become more common in recent years. Most column cross sections are round, although rectangular and square are used as well. Caps and columns are usually treated separately in design [6.1]. Foundations for cast-in-place columns are typically drilled shafts or pile footings. Foundations and single-column bents are not addressed in this research.

6.2.2 Bent Cap Analysis and Design

Rectangular bent caps are typically analyzed using an in-house bent cap analysis program, CAP18, developed in the mid-1960's [6.2]. CAP18 conducts a continuous beam analysis of the cap, assuming knife-edge (i.e., pin-connected) supports at the center of each column or pile. Shear and moment envelopes are generated from the analysis based on dead and live loads consistent with the AASHTO Standard Specification [6.3]. Loads associated with wind, longitudinal and centrifugal forces, and thermal expansions are not typically accounted for in bent cap design.

Other assumptions and guidelines for rectangular bent cap design include [6.1, Section 6.2]:

1. Class C concrete, with a design strength of 3600 psi
2. Cap depth at least as large as the cap width and sufficient to accommodate a reasonable pattern of tension reinforcement based on 60 ksi nominal yield strength
3. Stirrups between column faces based on shear envelope requirements and a shear strength of $2\sqrt{f'_c}$, where f'_c is the 28-day compressive strength of the concrete. In addition, #5 stirrups @ 6 in. are included in the overhanging ends of the bent cap, unless the distance from the center of load to the effective face exceeds 1.2 times the effective depth.
4. Longitudinal top and bottom bars based on moment envelope requirements and conservative estimates of bar lengths to satisfy development length requirements
5. Distribution of flexural reinforcement sufficient to limit cracking and to restrict service-level dead load stresses to 22 ksi
6. Side-face reinforcement to limit side-face cracking under service loads

7. Appropriate bar sizes, including #4 to #6 bars for stirrups and temperature reinforcement, and #9 to #11 bars for longitudinal reinforcement. Smaller longitudinal bars may be used for trestle pile caps. One bar size is normally used for all longitudinal bars.

Design of inverted-tee caps for multi-column bents requires checking six possible failure modes [6.4]. In addition to flexure and shear, cap design must consider: 1) torsion, 2) hanger tension in web stirrups, 3) flange punching shear at bearings, and 4) bracket failure at the flange-web interface. Development of wide cracks at the flange-web interface due to local stirrup yielding must also be checked at service-level loads. Features of inverted-tee design that differ from rectangular beam design include:

1. Ledge depth and reinforcement determined from punching shear, shear friction (bracket failure), or flexure
2. Web reinforcement accounting for hanger loads, vertical shear, and torsion
3. More stringent side reinforcement requirements
4. Consideration of torsion.

Section 5.13.2.5 of the AASHTO LRFD Bridge Design Specifications also addresses these issues. Figures 6.1 and 6.2, taken from Reference 6.1, show typical cap reinforcement for rectangular and inverted-tee caps.

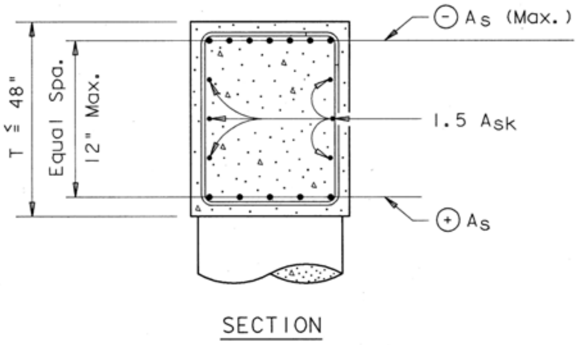


Figure 6.1 Typical Bent Cap Reinforcement—Rectangular Cross Section [6.1]

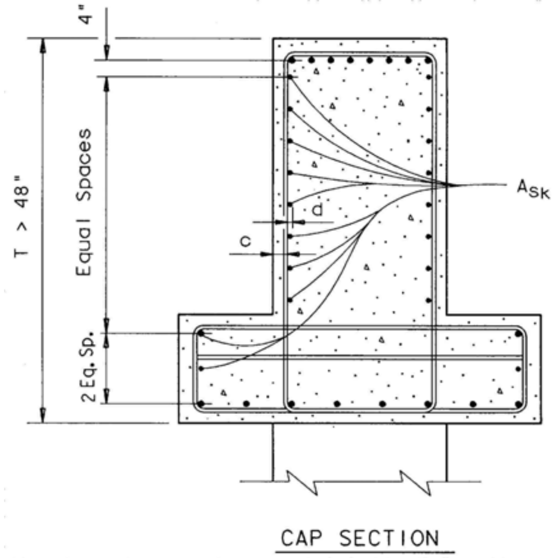
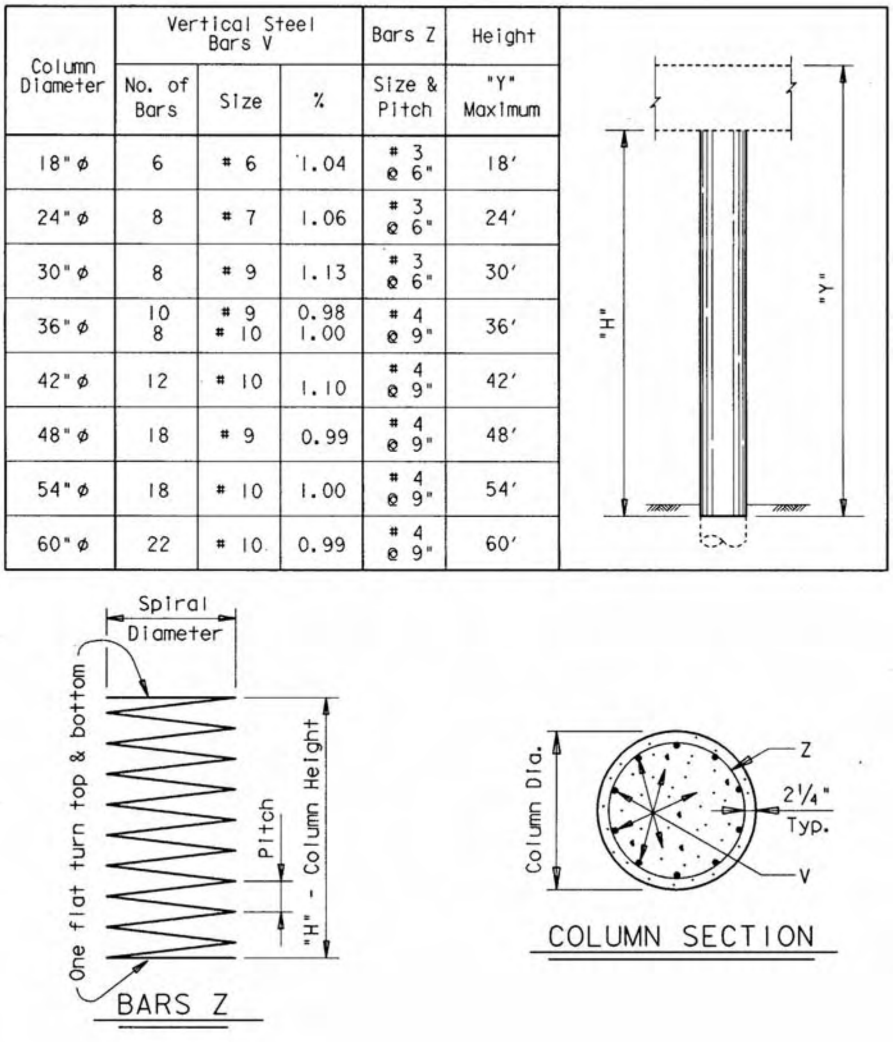


Figure 6.2 Typical Bent Cap Reinforcement—Inverted-tee Cross Section [6.1]

6.2.3 Column Analysis and Design

According to standard practice used in the TxDOT Bridge Design Division in Austin as well as other offices, column design for multi-column, single-tier bents does not normally involve analysis. As shown in Figure 6.3, round columns of standard sizes and reinforcement are normally used, with approximately one-percent longitudinal reinforcement and #3 spiral (6-in. pitch) or #4 spiral (9-in. pitch). Square or rectangular columns are also sometimes used. Based on a series of analyses conducted by TxDOT in the 1960's, these standard sections may be used without analysis if the column height satisfies a maximum height limitation of one foot per inch of column diameter [6.5]. The vast majority of bents satisfy this limitation.



Column Diameter	Vertical Steel Bars V			Bars Z	Height
	No. of Bars	Size	%	Size & Pitch	"Y" Maximum
18"ϕ	6	# 6	1.04	# 3 @ 6"	18'
24"ϕ	8	# 7	1.06	# 3 @ 6"	24'
30"ϕ	8	# 9	1.13	# 3 @ 6"	30'
36"ϕ	10 8	# 9 # 10	0.98 1.00	# 4 @ 9"	36'
42"ϕ	12	# 10	1.10	# 4 @ 9"	42'
48"ϕ	18	# 9	0.99	# 4 @ 9"	48'
54"ϕ	18	# 10	1.00	# 4 @ 9"	54'
60"ϕ	22	# 10	0.99	# 4 @ 9"	60'

Figure 6.3 TxDOT Standard Column Sizes and Minimum Reinforcement [6.1]

Some TxDOT offices do not use this "1 foot-per-inch" rule, but analyze bent frames for all cases. Even in offices that use this rule, analysis is required for columns that exceed this height limitation or for columns of unusual cross section, since such cases fall outside of the range of design parameters used to establish these guidelines. Factored axial load-moment combinations from frame analyses are compared to a design interaction diagram that is commonly generated using a proprietary program, such as PCACOL [6.6]. Column axial loads are sometimes taken as the reactions from bent cap analyses.

Horizontal loads due to wind, braking, and centrifugal forces are resolved into components in the transverse and longitudinal directions. Thermal effects are sometimes accounted for in analysis by the application of a horizontal displacement at the column top. This displacement is often limited to the maximum displacement associated with closing of expansion joints. Seismic effects are not considered in design. Frame analyses assume columns are fixed at the bottom, usually 5 to 15 ft below grade depending on subsurface soil conditions, and pinned at the top in the longitudinal direction ("flagpole" assumption). Full continuity is assumed at column tops in the transverse direction.

Second-order effects are normally accounted for by a simple moment magnifier from AASHTO Standard Specifications or using in-house programs [6.7,6.8]. Moments in the transverse and longitudinal directions for round columns are combined using the square root of the sum of the squares (SRSS) and then magnified, whereas moments for columns of non-circular cross section are first magnified in each direction and then combined using an interaction equation. Columns of non-circular section may also be checked using another in-house analysis program [6.9].

Trestle pile design is also based on standard height limitations of 1 foot-per-inch of pile depth. For piles exceeding these limitations, load effects at service and factored levels must be checked. Tensile and compressive stresses of the prestressed pile are restricted to the limiting values given in the AASHTO Standard Specifications. In addition, pile strength is checked using an axial load-moment interaction diagram at the factored level.

6.2.4 Standard Practice for Connection Design

The bent cap-to-column (or pile) connection region is the vertical extension of the column area into the bent cap. Reference 6.1 does not document an approach for the design of such connections. Standard design practice is limited to extending all column longitudinal bars into the cap a length equal to the cap depth minus 6 in. Although this length normally does not satisfy AASHTO development length requirements, no problems are known to have occurred in the field due to this standard practice. Piles are typically broken back sufficiently to extend strands 10 in. to 16 in. into the cap. Overhanging ends of caps provide room to anchor cap longitudinal reinforcement outside the joint region for exterior columns or piles. Explicit design requirements for bar anchorage, joint confinement, or joint shear are not used.

6.3 DESIGN APPROACH FOR A PRECAST BENT CAP SYSTEM

6.3.1 Design Philosophy

Design of a precast bent cap system should satisfy requirements of economics, constructability, durability, and force transfer. The design philosophy for force transfer is discussed in this section. Design of the system for force transfer must provide not only suitable strength, but also adequate ductility, redundancy and structural integrity.

TxDOT currently designs bridges according to the 1996 AASHTO Standard Specifications. In the future, TxDOT will adopt the AASHTO LRFD Bridge Design Specifications [6.10]. The design approach used herein not only satisfies the 1996 AASHTO Standard Specifications, but also addresses basic provisions of the 1998 AASHTO LRFD and select provisions of ACI 318-99 [6.11]. Judgment is used, however, to avoid imposing excessive requirements that could place the precast system at an unfair disadvantage with respect to cast-in-place bents.

6.3.1.1 Limit States

Four basic limit states are considered [6.10, Section 1.3.2]:

1. Strength—to ensure adequate strength and stability
2. Service—to restrict stress, deformation and crack width under regular service conditions

3. Fatigue and fracture—to restrict stress range due to a single design truck occurring at the number of expected stress range cycles
4. Extreme event—to ensure structural survival under extreme events such as vehicle collision, ship impact, or other occurrences.

Differences in design for cast-in-place and precast bents center on frame analysis and connection design. Phase 1-3 tests addressed strength and service limit states. Based on years of experience, TxDOT does not consider the fatigue and fracture limit to be a problem for cast-in-place bents, and thus it is not normally checked [6.5]. This is understandable, given the large service-load compression and small load eccentricities at the column or pile tops. The scope of this research did not allow high-cycle fatigue loads to be investigated. For cases in which a precast bent cap system is expected to behave in a manner significantly different than a cast-in-place bent (as discussed later), it is conservatively recommended that the fatigue and fracture limit state be explicitly checked. This may be important given the potentially small number of connectors that may be used in design.

The extreme limit state includes rare but conceivable events, such as vehicle collision and ship impact, whose return period may be much greater than the design life of the bridge. To mitigate potential collapse of bents due to vehicle collision or ship impact, TxDOT normally provides vehicle barriers or dolphins and usually designs multi-column bents with a minimum of three columns. This is in accordance with the intent of Section 1.1.2 of AASHTO Standard Specifications for structural integrity. AASHTO LRFD does not require design for vehicle collisions or vessel impact if adequate barriers, dolphins, or other sacrificial devices are used.

However, unforeseen events are possible. For example, thermal expansion of a cast-in-place slab superstructure caused a bent cap to rotate about the top of the piles, which were embedded in the cap. This resulted in a failure as the pile broke through the side of the cap [6.12]. Although such unexpected events are not normally designed for, minimum detailing requirements should be incorporated in the design procedure to permit load redistribution for unexpected loading.

6.3.1.2 Ductility

Ductility implies member proportioning and detailing that ensure significant inelastic deformations at the strength limit state prior to failure. Ductile behavior thus provides warning of impending failure. Brittle behavior of structural members and connections should be avoided where possible, as it implies sudden loss of load-carrying capacity. Suitable testing of specimens is typically required to verify ductile behavior [6.10, Section 1.3.3].

For a bridge system to achieve adequate inelastic behavior, it should have a number of members, joints and connections that are either ductile or have sufficient excess strength to assure inelastic response occurs at locations designed to provide ductile response. Section 1.3.3 of AASHTO LRFD indicates that ductility requirements are satisfied for a concrete structure in which the resistance of a connection is at least 1.3 times the maximum force effect imposed on the connection by the inelastic action of adjacent components. A load modifier of 1.05 is required by AASHTO LRFD for nonductile connections.

6.3.1.3 Redundancy

Multiple-load-path and continuous structures are desirable but not always possible in a bridge system. Main elements and components whose failure is expected to cause collapse are designated as failure-critical and the system non-redundant. AASHTO LRFD recommends that non-redundant systems incorporate a larger load factor than redundant members for the strength limit state. The columns, cap, and connections for an interior bent are all deemed failure-critical and thus the system is unavoidably classified as non-redundant.

6.3.1.4 Structural Integrity

Structural integrity includes redundancy but addresses additional issues. The overall integrity of a structure can often be significantly improved by minor changes in the detailing of members and connection hardware. In detailing connections, structural members should be tied together to improve redundancy and ductility. Thus, in the event of damage to a major supporting element or abnormal loading, damage may be limited to a relatively small area and the structure may be more likely to maintain stability [6.11, Section R7.13]. Section 1.1.2 of the AASHTO Standard Specifications requires that designs and details for new bridges should address structural integrity by considering:

1. Use of continuity and redundancy to provide one or more alternate load paths
2. Structural members that are resistant to damage or instability
3. External protection systems to minimize the effects of reasonably conceived severe loads.

Sections 7.13 and 16.5 of ACI 318-99 stipulate certain requirements for structural integrity, including the following tension tie requirements for precast structures (commentary shown in italics):

1. For precast concrete construction, tension ties shall be provided in the transverse, longitudinal, and vertical direction…to effectively tie elements together (Section 7.13.3). *Details should provide connections to resist applied loads. Connection details that rely solely on friction caused by gravity forces are not permitted. Connection details should be arranged so as to minimize the potential for cracking due to restrained creep, shrinkage and temperature movements.*
2. Vertical tension tie requirements of 7.13.3 shall apply to all vertical structural members…and shall be achieved by providing connections at horizontal joints in accordance with the following:

   Precast columns shall have a nominal strength in tension not less than $200A_g$ in pounds. For columns with a larger cross section than required by consideration of loading, a reduced effective area A_g, based on cross section required but not less than one-half the total area, shall be permitted. (Section 16.5.1.3)…*Connections at horizontal joints in precast columns…are designed to transfer all design forces and moments. The minimum tie requirements are not additive to these design requirements.*

Reference 6.13 indicates that the foregoing requirement for force transfer is essentially a resistance of 200 psi times the gross area.

6.3.1.5 Design Provisions for Ductility, Redundancy, and Structural Integrity

Various sections of Chapter 6 introduce conservatism into connection design to provide for ductility, redundancy, and structural integrity. Provisions for ductility include: 1) use of connection actions (i.e., forces and moments) based on conservative assumptions in analysis, 2) 1.3 times the factored loads for design of connector steel area, 3) conservative derivation of development length equations, including the use of 1.25 times the specified yield strength, and minimum required embedment depths, and 4) requirement for minimum area of confining reinforcement.

Provisions for redundancy include the use of: 1) minimum requirements for the number of connectors, 2) optional use of headed reinforcement, and 3) optional use of bolted connections. Structural integrity includes provisions for redundancy as well as an additional minimum requirement for connector steel area.

6.3.2 Design Flow Chart

This section provides an overview of the primary steps used in the design of a precast bent cap system, based on the principles outlined in the previous section and test results from Phases 1-3. This design methodology applies only to the most common multi-column and trestle pile bents and is not intended to apply to single-column bents, bents subjected to seismic or other highly dynamic loads, or bents of unusual proportions or applications. The flow chart shown in Figure 6.4 outlines the eight steps in the design procedure. This procedure is briefly summarized below. A detailed discussion of each step is provided in subsequent sections.

Design begins with selection of a trial bent configuration, including cap and column cross sections and layout. A preliminary cap depth may be based on an estimate of the development length for an assumed connector size (e.g., the size used for a similar cast-in-place cap). The bent cap will normally be analyzed and designed using CAP18, and standard column details may be used where predetermined height limitations are satisfied. Non-circular columns or columns exceeding height limitations require analysis. The bent configuration may be modified as necessary. The final bent configuration is then analyzed to determine connection design actions in both the transverse and longitudinal directions. Full beam-column continuity is conservatively assumed in determining connection actions. A planar frame analysis is required for transverse actions, even if standard column sections are used.

The connection type is then selected based on design actions as well as considerations of constructability, durability, and economics. The designer also decides if the column or pile will be embedded into the cap or built surface-flush (i.e., flush with the cap soffit). Then, a trial connector configuration is chosen, including the number, size, yield strength, and arrangement of connectors. Based on this configuration, limit states are investigated. For the strength limit state, axial load-moment design strength accounting for biaxial bending in the transverse and longitudinal directions is checked, as well as shear friction and joint shear. Serviceability checks include potential opening at the bedding layer, cracking at the cap top, and deflections. Response at the extreme limit state are checked if barriers are not provided. Based on these analyses, the connector configuration or even connection type may be modified.

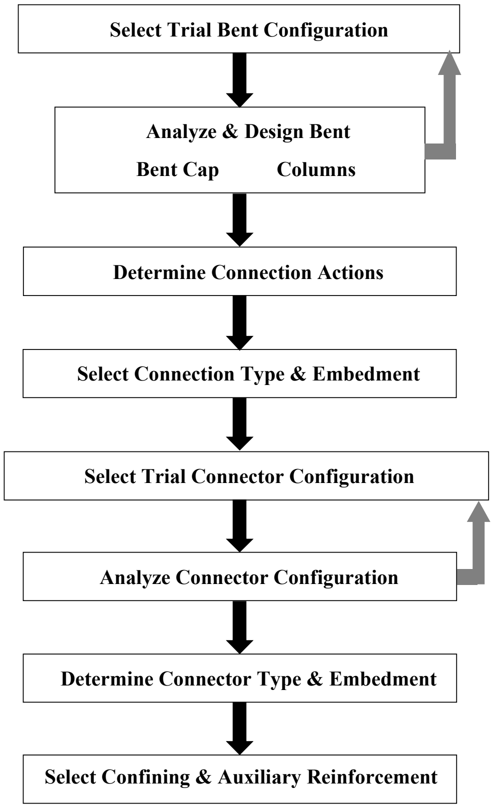


Figure 6.4 Design Flow Chart for Precast Bent Cap System

The connector anchorage (straight or headed bars) and coating (uncoated or epoxy-coated bars) are then selected. For grout pocket and grouted vertical duct connections, connector embedment depths in the cap and column or pile are determined. Finally, confining reinforcement and auxiliary reinforcement for the connection are determined.

6.3.3 Selection of Trial Bent Configuration

The first step in the design process is to select a trial bent configuration, including the cap type (rectangular or inverted-tee), cross-sectional dimensions, and length, as well as the number, spacing, and size of columns or piles. Column spacing is typically 16 ft to 20 ft for rectangular caps and approximately twice as large for inverted-tee caps. Pile spacing is approximately equal to beam spacing of piles. Girder type, span lengths, and superstructure definition will have some effect, but usually will not drive the design except for inverted-tee caps.

Bent configurations can affect selection of the connection type. For example, all connection types should usually be feasible for rectangular caps, whereas grout pockets may be difficult to form within an inverted-tee cap, as explained in Chapter 2. The designer may use development length equations (Section 6.3.9.3) to estimate the required cap depth if grout pocket or grouted vertical duct connections are likely to be used. Maximum cap dimensions and/or length may be limited by crane capacities, especially for inverted-tee caps, which will normally weigh more than 2.5 kips/ft. In many cases, an economical design may require that caps not weigh more than the heaviest beam.

6.3.4 Analysis and Design of Bent Cap and Columns

6.3.4.1 General Connection Behavior

Proper modeling of a bent cap and columns for analysis and design depends on actual connection behavior. Testing described in Chapters 3 to 5 demonstrated that use of a relatively small number of connectors (4) not only facilitated construction but also provided sufficient strength, ductility, and stiffness for service and factored loads at realistic eccentricities. Strengths were predicted within 15 percent in Phase 2 and 20 percent in Phase 3, with the failure load exceeding the predicted capacity for all but one case (CDL4 in Phase 2). Ductility was also exceptional: connectors not only achieved yield, but testing was discontinued in many cases due to large connector strains exceeding 1 percent and correspondingly large cap deflections. Also, inelastic action was not limited to the bedding layer. Phase 2 strain records demonstrated connector yield spreading as far as half the embedment depth (i.e., 7.5 in.) into connections without loss of anchorage. Crack widening in the pile and columns also indicated excellent transfer of forces through the connection, as well as ductility. At realistic eccentricities, there were no openings at the bedding layer. Despite the potentially small rotational stiffness, deflections were small because of the small moment transfer. Connection stiffness is addressed in the next section. An additional benefit of using fewer connectors is the relief of potential stresses due to temperature and volume changes [6.14, Section 6.11]. Behavior indicated that a precast bent cap system can use fewer connectors (e.g., 4 to 8 connectors) than a cast-in-place bent (see Section 6.3.7).

As discussed in Section 1.5.4.2 and Chapter 4, if a precast bent cap system were to use a similar number of connectors as for a cast-in-place bent (e.g., 8 to 12) with a similar distribution around the perimeter, then differences in structural performance for a precast system could be caused by: 1) the presence of a grouted bedding layer, 2) anchorage of connectors in a grouted connection, and 3) potential embedment of the column or pile into the cap. Based on Phase 2 and 3 tests, it was concluded that the existence of a bedding layer up to 4 in. thick is not expected to significantly affect connection response. Phase 1-3 tests also demonstrated excellent anchorage of connectors with minimal slip. The influence of column or pile embedment is unknown, but is expected to provide a connection stiffness at least equal to that of a surface-flush condition (like that used in a cast-in-place bent). Thus, these three factors are not expected to influence connection response. As mentioned in Section 1.6.2.3, others have found a grouted bedding layer to have little or no effect on behavior compared to cast-in-place specimens and with anchorage of all the column bars, monolithic behavior can be emulated [1.31,1.33, 1.36].

In addition, Stanton et. al. found that when a small number of connectors were grouted in ducts, strength was accurately predicted by conventional reinforced concrete analysis and excellent ductility resulted [1.32]. Concern was expressed regarding anchorage of reinforcing bars in relatively shallow beam depths of typical buildings if full-moment transfer is desired. However, Phase 1-3 tests demonstrated that for typical bent cap depths, adequate anchorage is feasible for large diameter bars (e.g., #11's) so that large moment transfer is possible.

6.3.4.2 Rotational Stiffness of Connections

Because differences in structural behavior for a precast vs. cast-in-place system are not likely to be caused by the bedding layer or connector anchorage, the major factor that can produce a difference is the number and location of connectors. The expected result of using a smaller number of connectors is a smaller rotational stiffness of the connection. This was exhibited in Figure 4.92 for Phase 2 tests, which showed that a connection using a 2/2 configuration (two rows, two connectors per row; connector reinforcement equal to 0.57 percent of column area) produced a bent deflection approximately twice that predicted from analysis using a rigid connection (prior to connector yield). Using a mix of different connections and connector configurations, Phase 3 bents subjected exhibited beam deflections within approximately 30 percent of a frame analysis assuming rigid connections (for loading with a transverse eccentricity). Eccentricities in Phase 3 were much smaller than for Phase 2. It was also demonstrated in Chapter 4 (Figure 4.93) that the actual section stiffness associated with moment-curvature response was predicted reasonably well using a sectional analysis. This suggests that the analytical moment-curvature response for a given connector arrangement at the column top can be used to establish the connection rotational stiffness for frame analysis. Frame analysis using a reasonably accurate rotational spring stiffness at the connection would then be expected to produce realistic design forces and deflections.

Figure 6.5 compares the moment-curvature response predicted for a hypothetical 30-in. column (3600-psi concrete, #9 Grade 60 connectors) using 2/2, 2/3, and 2/4 connector arrangements, as well as a cast-in-place configuration. Both transverse and longitudinal response is shown for unsymmetrical connector arrangements. Figure 6.6 shows the neutral axes at failure (assumed to correspond with a compressive strain of 0.004). It is evident from this figure that as the number of connectors increases, the section stiffness (slope of the moment-curvature plot) also increases. However, this increase is not proportional to the increase in percentage of connector steel, because connectors must be located within relatively narrow confines in the column core. For example, a 2/4 configuration uses twice the connector reinforcement (1.13 percent) as a 2/2 configuration (0.57 percent), but achieves an increase in section stiffness (prior to yield) of only two-thirds.

Comparison to a cast-in-place configuration is also important, because a rigid joint is normally assumed in frame analysis of bents. This assumption has normally produced acceptable designs. Cast-in-place bents achieve continuity at the connection by the extension of all column bars into the cap. For many bents used in Texas, this often involves the extension of one percent longitudinal reinforcement (Figure 6.3).

Figure 6.5 shows that as the connector percentage increases, the section stiffness approaches that of a cast-in-place configuration. A 2/2 configuration has a stiffness 45 percent of that for a cast-in-place configuration. For both the transverse and longitudinal directions, a 2/4 configuration exhibits a stiffness that is approximately 75 percent of that for a cast-in-place configuration. For realistic connector configurations it is unlikely that a precast bent cap connection can achieve the same stiffness as a cast-in-place system.

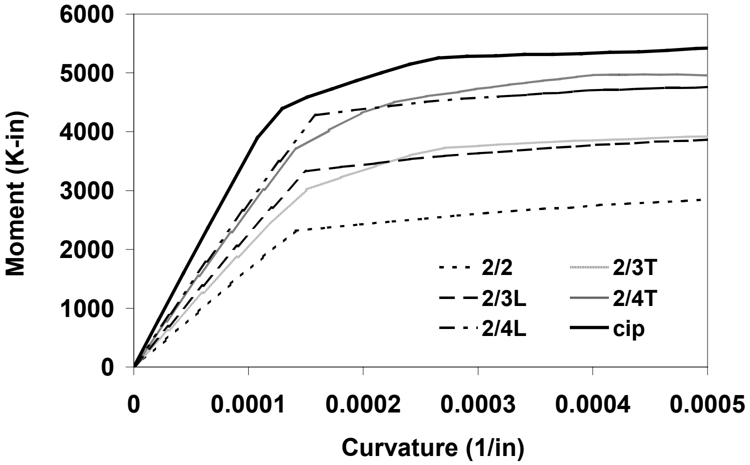
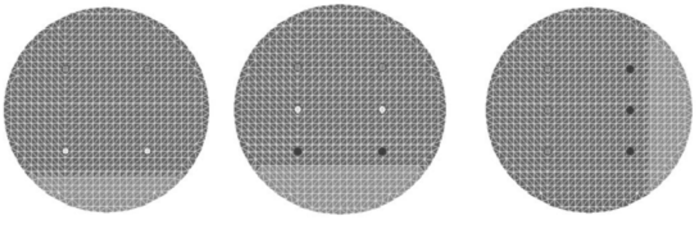
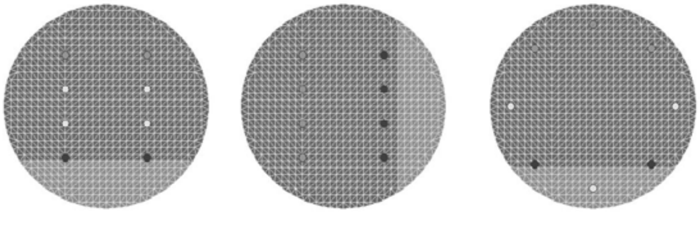


Figure 6.5 Predicted Moment-Curvature Response for Connector Configurations



2/2 2/3 Transverse 2/3 Longitudinal



2/4 Transverse 2/4 Transverse Cast-in-Place

Figure 6.6 Predicted Neutral Axis Locations for Various Connector Configurations

Although connection stiffness is related to section stiffness, it is defined by the slope of the moment-rotation relationship. Figure 6.7 shows the moment-rotation relationship for two of the CBC specimen tests. The entire load history is shown for CBC1, but a truncated plot is shown for CBC4 due to anomalies in the strain gage records at larger loads. The deformations at the SE and SW connectors was calculated based on strain gage measurements at the 1 in. location above and below the bedding layer. These deformations were then used to calculate the rotation at the bedding layer. The CBC1 specimen was loaded with a 4.25-in. transverse eccentricity, whereas the CBC4 specimen was loaded with a failure eccentricity of 55.5 in. The connection stiffness for CBC4 is approximately 1/5 that of CBC4. This is due to prior loading of the CBC specimen to factored levels. The nearly vertical slope of CBC1 indicates that at small eccentricities the connection stiffness was large. This explains, in part, why frame analyses assuming a rigid joint predicted specimen deflections for the first Phase 2 test more closely than for later tests. Deflections for Phase 3 specimens, which used small eccentricities, were also predicted closely.

Thus, establishing realistic values of the rotational spring stiffness for specific conditions is not a simple task. The rotational stiffness may depend on the connection configuration, level of loading, and load-history, among other variables. Additional tests and analysis would be required to develop a model for the rotational stiffness. For this, and other reasons, a simpler approach is needed for design.

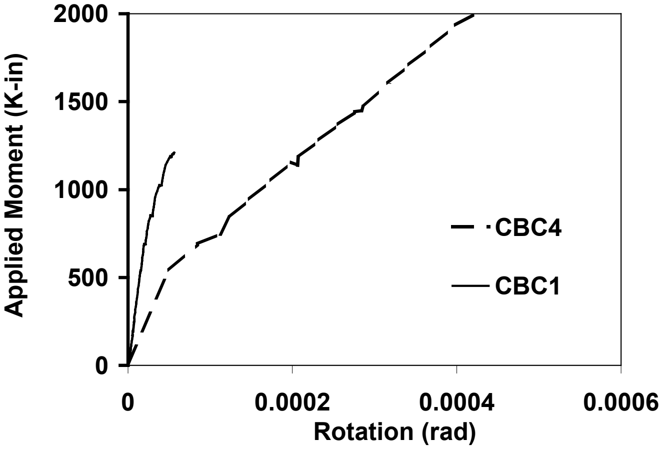


Figure 6.7 Moment-Rotation Relationship for Phase 2 CBC Specimen

6.3.4.3 Design Implications of Connection Stiffness

The design strategy for a precast bent cap system is to provide sufficient strength, ductility, and stiffness to satisfy the limit states, but not to emulate behavior of a cast-in-place bent. Practically, this will result in a smaller stiffness at the connection than for a cast-in-place bent. Since connections for a precast bent cap system are semi-rigid, i.e., having a stiffness between that provided by a cast-in-place system (taken as "rigid" for analysis) and a pin (i.e., no rotational restraint), a simple yet conservative design approach is to simply check the system response for these two limiting cases and design for the governing case. Little additional effort is required when using a computer model to compare response for the bounding cases. This approach is recommended until further research is conducted to establish realistic connection stiffness.

Therefore, the implications of the actual connection stiffness on design may be summarized as follows:

1. Designers may determine the bounds of in-plane bent response by conducting two analyses: one assuming a rigid connection between the cap and column, and the second assuming a pin. Connection design is then based on the worst-case response. Design of the bent cap and columns can also be based on these design actions (see subsequent sections for details). This approach eliminates a significant amount of potential iteration associated with varying connection stiffness for each connector configuration.
2. If connection stiffness can be realistically established for different connection configurations and types, designers can use this in frame analysis. An iterative procedure would be required.

6.3.4.4 Bent Cap Analysis

Strictly speaking, design of a bent cap should be based on the two bounds of response as mentioned in the previous section. However, TxDOT currently designs bent caps using an in-house bent cap analysis program, CAP18, that assumes pinned connections at column tops (i.e., continuous beam analysis). Although this approach neglects moment transfer, it has resulted in successful design of bent caps for decades. Since conditions for a precast connection more closely match the pinned assumption than do conditions for a monolithic connection, CAP18 can still be used for design of a precast bent cap.

6.3.4.5 Column Analysis

As mentioned in Section 6.2.3, the vast majority of columns (and piles) in single tier bents satisfy the "1 ft.-per-in." rule and therefore are not analyzed by some TxDOT offices. For cases requiring analysis, columns are assumed to be pinned at the top in the longitudinal direction (flag-pole assumption), but to maintain full continuity with the cap in the transverse direction. Pinned connections at the column tops for both directions were conservatively assumed in column studies that became the basis for the "1 ft.-per-in." rule [6.5].

Because the assumptions incorporated into the "1 ft.-per-in." rule are expected to be reasonably consistent with the conditions of a precast bent cap system, design of columns meeting the height limitations do not require analysis in offices that have adopted the "no-analysis" column design. However, for columns requiring analysis, the designer should analyze the bent using a pin and rigid connection in the frame model to bound the response and then select the governing actions for design. In either case, a frame model is required for all bents to determine connection actions (see next section). With experience, the designer will be able to quickly determine cases for which column design is appreciably affected by the different joint continuity.

6.3.5 Determination of Connection Design Actions

Because of the critical function of connections in a precast bent cap system, connection actions should be conservatively determined. As for column design, it is important that connection actions in both the transverse and longitudinal directions be considered. For transverse response, a planar frame analysis should be conducted to determine connection actions. As mentioned previously, in lieu of a reasonably accurate moment-rotation relationship, connection actions can be conservatively determined by assuming a rigid joint at the column top, which will maximize connection design forces and eliminate iterations in design. This frame model is the same as that used for column analysis.

For relatively short columns, transverse moment requirements may be governed by dead and live loads. For taller columns, wind is likely to be included in the governing load combination. Calculation of maximum longitudinal moments will typically require patterning live loads on one of the spans adjacent to the bent. Forces associated with thermal effects, joint closing, and braking may also produce significant longitudinal moments and should be accounted for. Longitudinal moments at an individual column may be determined from static equilibrium: current TxDOT standard practice is to assume all columns resist an equal fraction of the total longitudinal moment.

The governing load combination for connection design depends on the most severe combination of simultaneous transverse and longitudinal loading. It is possible that the most severe transverse loading condition may only produce moderate load effects in the longitudinal direction, and visa-versa. For example, wind plus dead load may govern in the transverse direction, but produce negligible effects on the connection in the longitudinal direction because full wind load is never in combination with longitudinal forces. However, wind and thermal loads are in combination and could thus govern the design. It is also possible that the AASHTO Standard Specifications Group VI load combination (dead, live, partial wind, wind on live load, longitudinal, and thermal loads) may govern. This may not produce the most severe condition in the transverse direction, but the combined effects for both directions may be largest. In addition, adoption of AASHTO LRFD for design will result in different load combinations being used that may govern design. For example, live loads (lane load plus design truck) and braking loads are significantly larger in the LRFD code than in the Standard Specifications. Also, LRFD load factors and load types within a combination differ from those required by the Standard Specifications. The extent to which this will affect design is not yet known.

Columns should be able to achieve their capacity prior to a connection failure. This requires that the connection have sufficient excess capacity to handle the actions associated with the development of the actual column capacity. Since columns will be proportioned with some excess capacity beyond the factored design actions, the connection must be able to the resist additional loads associated with this extra capacity. The extra 1.3 factor applied to connection design actions, as specified in Section 6.3.1.5, addresses this concern.

6.3.6 Selection of Connection Type and Embedment

After determining the connection actions, the designer selects the connection type and embedment. Four connection types have been considered in the research: 1) grout pockets, 2) grouted vertical ducts, 3) grouted sleeve couplers, and 4) bolted connections. Two embedment configurations for columns or piles have been investigated: surface-flush and embedded.

6.3.6.1 Connection Type

6.3.6.1.1 Comparisons

Selection of a connection type will typically depend on a number of factors including economics, constructability, durability, and force transfer. The relative importance of each factor may vary for specific projects. Section 2.2 discussed advantages and disadvantages related primarily to constructability and durability for each connection type. Major uncertainties for grout pocket, grouted vertical duct, and bolted connections were addressed in Phases 1-3. Based on Section 2.2 and Phase 1-3 test results, Table 6.1 provides a qualitative comparison of connection types in terms of constructability, durability, and force transfer. An accurate assessment of economics awaits system implementation.

Table 6.1 shows that constructability advantages and disadvantages vary widely for the different connection types. All connection types except grouted sleeve couplers can be designed to provide ample construction tolerances. The extremely tight tolerances, proprietary hardware and grout, and required grout pumping cancel in large measure the durability and force transfer advantages of grouted sleeve couplers. Grout pockets may be tailored advantageously for constructability and force transfer and use simple grouting operations. However, they may be more susceptible than other connection types to durability problems associated with service-level cracking in the connection region (i.e., pocket). Grout pockets should also be carefully designed to limit congestion of longitudinal cap reinforcement. This is particularly important for single-line grout pockets, which are susceptible to congestion and large spacing of the cap longitudinal reinforcement.

Grouted vertical ducts appear to possess significant advantages in constructability, durability, and force transfer. Adequate tolerances are provided by sizing ducts approximately 3 to 4 times the connector diameter, but not less than 3.5 in. Although tremie-tube grouting may be more challenging than the bucket method permitted for grout pockets, successful grouting is expected if precast connection specifications are followed. Based on Phase 1-3 tests, grouted vertical ducts are also expected to provide excellent protection and anchorage of connectors.

Bolted connections provide all the major advantages of grouted vertical ducts. In addition, bolted connections accommodate an alternative cap setting approach using leveling nuts and plates, provide temporary support during erection, and allow for post-tensioning. Larger moment transfer may be accommodated through the use of high strength bolts, and redundancy is provided by bond and cap top anchorage. One disadvantage of a bolted connection is the cap top anchorage, which will require special encasement or recessed pockets that are grouted to prevent potential durability problems. Conflict between cap top anchorages and beam bearing seats must also be avoided.

Table 6.1 Advantages and Disadvantages of Connection Types

Connection Type	Constructability	Durability	Force Transfer
Grout Pocket	+large construction tolerances +tailored pocket shapes +easy to place confining reinforcement +fairly simple grouting operations -congestion of reinforcement -large spacing between reinforcement	+epoxy-coated connectors viable -cracking at large top surface -cracking through connectors	+simple to tailor number of connectors roughening +excellent anchorage of connectors +excellent ductility of connectors +relatively simple anchorage design approach -potentially small rotational stiffness
Grouted Vertical Duct	+acceptable construction tolerances +inexpensive stay-in-place ducts +minimal interference with cap reinforcement +easy to place confining reinforcement -more difficult grouting operations required	+more limited exposed top surface +well-protected connectors +epoxy-coated connectors viable	+excellent interlock at all interfaces +excellent anchorage of connectors +excellent ductility of connectors +simple anchorage design approach -potentially small rotational stiffness
Grouted Sleeve Coupler	+minimal interference with cap reinforcement -excessively tight horizontal tolerances -proprietary hardware and grout -higher level of construction skill required -grout pumping required in all cases -multiple grouting operations required	+well-protected connectors +epoxy-coated connectors viable	+excellent anchorage of connectors +excellent ductility of connectors +anchorage design not required for cap -potentially small rotational stiffness
Bolted Connection	+acceptable construction tolerances +inexpensive stay-in-place ducts +minimal interference with cap reinforcement +easy-to-place confining reinforcement +cap setting option using leveling nuts & plates +temporary support during erection +optional post-tensioning -more difficult grouting operations required -multiple grouting operations possibly required	+galvanized connectors viable +optional post-tensioning -grouting of cap top recess -exposed cap top anchorage	+resistance to large moments possible +full continuity of bars through connection anchorage +excellent ductility of connectors +anchorage design not required for cap -potentially small rotational stiffness

6.3.6.1.2 Design Guidelines for Grout Pocket Connections

The following guidelines are recommended for design of grout pocket connections:

1. *Pocket size*: The designer should strive for the smallest pocket size that provides sufficient horizontal tolerance. Required tolerances of at least 1 in. in the longitudinal direction and 2 in. in the transverse direction (to the centerline of the bridge) are expected. These tolerances should account for combined tolerances associated with placement of connectors in piles or columns and fabrication and placement of pockets and ducts in the bent cap. However, excessively large pockets should be avoided. The exposed top surface of pockets should be minimized for durability reasons. In addition, a relatively small grout pocket will limit congestion of typical bent cap reinforcement, including minimum reinforcement required for structural integrity, and will reduce the grout volume as well as the time required to conduct grouting operations. The grouting method should also be considered. A bucket approach will normally require that grout pockets extend through the entire cap depth. Grout ports into the pocket can be used where it is desirable (e.g., for durability reasons) to use a pocket height less than the full depth of the cap. Grout pockets are expected to be more suitable for use with rectangular caps rather than inverted-tee caps.
2. *Pocket shape*: Trapezoidal pocket shapes incorporating flat sides are recommended to simplify formwork design and form removal. Roughened surfaces are not necessary when minimum pocket tapers are used.
3. *Pocket taper*: Based on Phase 1-3 tests, a minimum taper of approximately two degrees in both the longitudinal and transverse directions is recommended for grout-concrete interlock and for ease in form removal. Single-line grout pockets can use a larger taper through the cross section of up to approximately four degrees. However, the designer should carefully consider the impact of a large taper through the cross section on congestion and spacing of longitudinal bars. Larger tapers are more easily accommodated along the cap, rather than through the cross section. “Keystone” tapers such as that shown in Figure 2.3 are feasible if a larger taper is desired, but will likely require stay-in-place forms.
4. *Double-line vs. single-line*: Single-line grout pockets can provide greater horizontal tolerances, but prevent the use of longitudinal reinforcement along the centerline of the cap and thus increase longitudinal bar spacing. When double-line grout pockets are used, longitudinal reinforcement may be positioned between the grout pockets. Double-line grout pockets provide connection to transfer forces and also provide a means of connector redundancy for trestle pile bents that require only a single-line grout pocket.

6.3.6.1.3 Design Guidelines for Grouted Vertical Duct and Bolted Connections

The following guidelines are recommended for design of grouted vertical duct and bolted connections:

1. *Duct diameter*: The designer should provide a horizontal tolerance of at least 1 in., although a minimum of 1.5 in. is preferable, especially for trestle pile bents. For a straight #11 bar, a minimum duct diameter of 3.5 in. should be used. Phases 2 and 3 demonstrated that a 4-in. duct diameter is preferable, even with a #9 straight bar. Duct diameters should generally be about 2 to 3 times the bar diameter. Diameters for standard steel-corrugated ducts are normally limited to approximately 5 in. and available in 1/8-in. or ¼-in. increments. It is recommended that duct diameters not exceed 5 in., to minimize interference with typical cap reinforcement, reduce the volume of on-site grouting, and because of the lack of test data for ducts larger than 4 in. For most projects, this size will satisfy #11 upset-headed bars with adequate tolerance. Larger headed bars may not provide sufficient tolerance.

2. *Duct height*: To facilitate grouting operations, ducts should normally extend the entire cap depth. For very deep inverted tee caps, ducts may be either partial-depth or full-depth. Partial-depth ducts may be gravity-flow grouted through grout tubes in the cap sidewall or pressure grouted from the bedding layer. Designers should provide a duct length at least three inches more than the required development length to accommodate vertical tolerances, adequately anchorage ducts, and ensure grout completely encompasses the connector. Duct anchorage was found to be excellent in all tests. In addition, duct strains were very small at locations beyond the bar embedment depth. The minimum connector embedment depth requirement of Section 6.3.7 should ensure no problem with duct anchorage.
3. *Duct material*: Semi-rigid, steel-corrugated ducts should be used. Ducts should be also be spirally-crimped with grout-tight joints. Material should be galvanized, cold-rolled steel per ASTM A619 and ASTM A527. Ducts should provide a corrugation height of at least 0.094 in. Duct thickness should be at least 26 gage (0.23 in.) for duct diameters up to 4.5 in. and 24-gage for duct diameters greater than 4.5 in. These provisions will help ensure: 1) adequate bond at interfaces between the concrete, duct, and grout, 2) suitable stiffness for handling and fabrication, and 3) corrosion protection. Plastic ducts were not tested and thus cannot be recommended. Low bond strength is expected.

6.3.6.2 Connection Embedment

Selection of the trial connector configuration also includes a decision regarding the embedment configuration. The surface-flush configuration may be simpler for cap setting and allows inspection of the bedding layer after grouting. However, the bedding remains exposed to the environment. Opening of the bedding or formation of voids at the bedding layer due to water or air pockets during grouting may lead to durability problems if preventive measures are not taken. Embedded columns or piles more effectively limit moisture paths into the connection than surface-flush connections and are thus expected to provide better durability protection for connectors. In addition, as shown in Phase 2 and 3 grouting operations, grout for embedded columns always develops a solid interface between the cap and column. One drawback of an embedded column or pile is the more limited access for post-grouting inspection. See Section 6.3.8.3.1 for additional options to enhance durability.

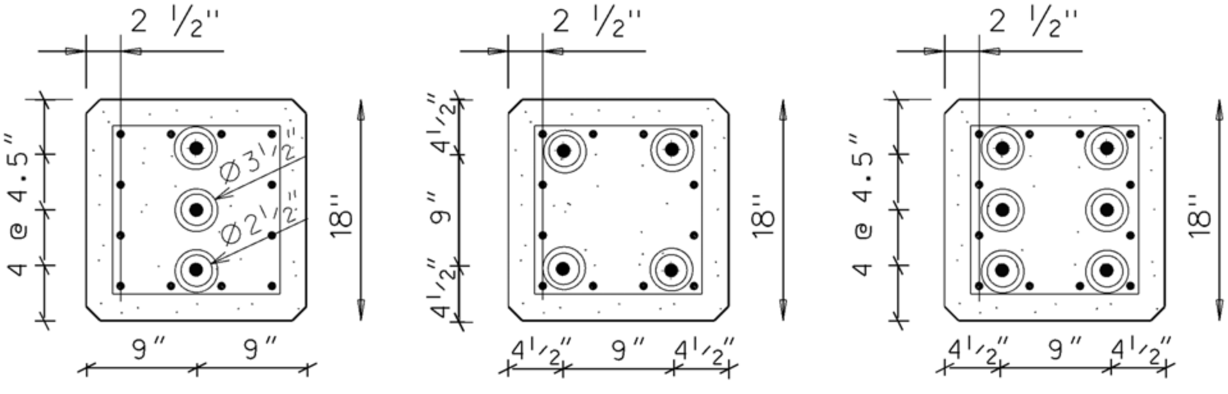
When columns or piles are embedded, an embedment depth between 2 in. and 5 in. is recommended to accommodate vertical tolerances. This range is expected to be sufficient to ensure complete embedment in the field and to provide durability protection. In addition, rotational stiffness of the connection is expected to be enhanced. An embedment of 3 in. to 5 in. is likely to be used for applications where corrosion is a major concern. The designer should be aware that embedment of the column or pile impacts placement of the bottom longitudinal reinforcement and reduces embedment depth of connectors. Reinforcement in shallower caps such as pile caps are more likely to be affected by a reduction in available embedment depth.

6.3.7 Selection of Trial Connector Configuration

After the connection type is selected, a trial connector configuration is chosen, including the number, size, yield strength, and arrangement of connectors. Because TxDOT uses standard column and pile sizes, cross-sectional dimensions are predefined and one of a fairly limited number of connector arrangements will likely be selected.

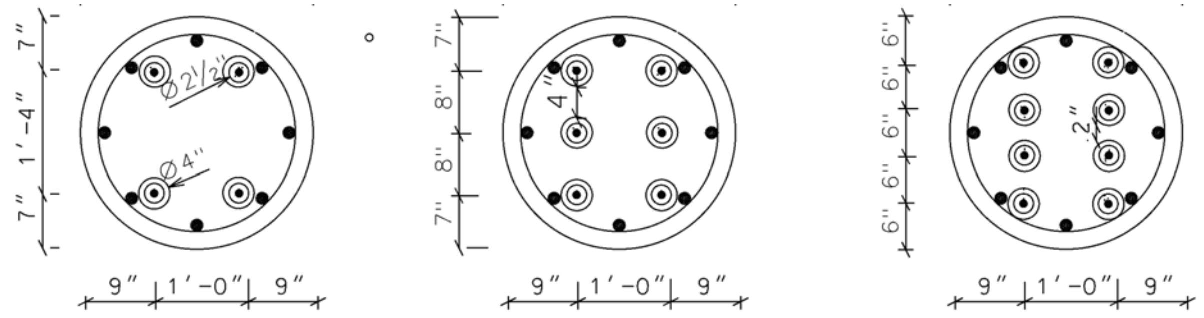
6.3.7.1 Example Configurations

Example connector configurations are shown for an 18-in. square pile and a 30-in. round column in Figures 6.8 and 6.9. Similar possibilities exist for other column sizes and shapes. Figure 6.8 shows three connector configurations for an 18-in. pile: a 1/3 (one row, three connectors), 2/2 (two rows, two connectors per row), and 2/3 (two rows, three connectors per row) options. If #9 bars are used, the percentage of connector cross-sectional area relative to the pile area is 0.93 percent, 1.23 percent, and 1.85 percent respectively (Table 6.2). Considerations other than force transfer may govern actual connector configurations. For example, the relatively small force transfer requirements for connections in a trestle pile bent may be readily satisfied by a 1/3 configuration. However, the designer may choose a 2/2 configuration to provide additional redundancy. In addition, constructability considerations may justify the use of a 2/2 configuration. Symmetry, less congestion, a better distribution of longitudinal reinforcement, and greater spacing between ducts (for vertical duct and bolted connections) may significantly simplify construction and make a 2/2 configuration a better choice. Force transfer requirements for a pile are rarely expected to exceed the capacity provided by a 2/3 configuration with #9 bars.



1/3 2/2 2/3

Figure 6.8 Example Connector Configurations—18-in. Pile



2/2 2/3 2/2/4 Longitudinal

Figure 6.9 Example Connector Configurations—30-in. Column

Figure 6.9 shows the configuration for a 30-in. round column. Two lines of connectors are shown for all cases, as more significant force transfer in both the longitudinal and transverse directions is expected for column bents. Table 6.2 shows that, for a 30-in. column, all configurations except a 2/2 with #9 bars provide at least 0.85 percent connector reinforcement. The 2/4 configuration provides a 2-in. spacing between the 4-in. ducts in the cap. For a 42-in. column, #11 connectors are likely to be used. A 2/3 arrangement on a 42-in. column provides 0.68 percent reinforcement. Additional capacity can be provided by using connectors of a larger number and/or size. As the number of connectors increases, additional redundancy is built into the system. Similar connection configurations exist for bolted connections.

Table 6.2 Connector Reinforcement Percentages

Column or Pile Size	Connector Reinforcement number-size (%)			
	1/3	2/2	2/3	2/4
16-in. Square Pile	3-#8 (0.93) 3-#9 (1.17)	4-#8 (1.23) 4-#9 (1.56)	N/A	N/A
18-in. Square Pile	3-#8 (0.73) 3-#9 (0.93)	4-#8 (0.98) 4-#9 (1.23)	6-#8 (1.46) 6-#9 (1.85)	N/A
24-in. Round Column	N/A	4-#9 (0.88) 4-#11 (1.38)	6-#9 (1.33) 6-#11 (2.07)	N/A
30-in. Round Column	N/A	4-#9 (0.57) 4-#11 (0.88)	6-#9 (0.85) 6-#11 (1.32)	8-#9 (1.13) 8-#11 (1.77)
36-in. Round Column	N/A	4-#9 (0.39) 4-#11 (0.61)	6-#9 (0.59) 6-#11 (0.92)	8-#9 (0.79) 8-#11 (1.23)
42-in. Round Column	N/A	4-#9 (0.29) 4-#11 (0.45)	6-#9 (0.43) 6-#11 (0.68)	8-#9 (0.58) 8-#11 (0.90)

The designer should account for the fact that the capacity in the transverse and longitudinal directions will differ when non-symmetrical connector configurations such as 2/3 and 2/4 configurations are used. The designer should also note that an increase in the number of connectors does not produce a proportional increase in axial load-moment capacity, especially for compression-controlled designs (i.e., small eccentricities). In addition, connector and duct spacing becomes more critical for construction and anchorage as the number of connectors increases, as discussed in the following section.

6.3.7.2 Minimum Spacing for Connectors and Ducts

6.3.7.2.1 Connector Spacing for Grout Pocket Connections

In selecting a connector configuration, the designer should attempt to maximize connector and duct spacing to facilitate constructability and to limit potential effects of splitting. It is recommended that a minimum clear spacing of $2d_b$ be used for straight bar connectors in grout pocket connections. This provides reasonable clearance for grouting and is expected to provide sufficient resistance to splitting.

Although the scope of testing prevented a detailed investigation of anchorage for bar groups in grout pockets, tests did provide some basis for a minimum clear spacing of $2d_b$. In Phase 1 pullout tests, cracks typically developed between connectors and the surrounding concrete. However, anchorage failure was related to loss of confinement of the surrounding concrete rather than splitting between connectors or between connectors and the concrete surface. Tests that used #8 headed bars with a $7d_b$ clear spacing exhibited some splitting between bars but such cracks did not alter the failure mode. In Phase 2 and 3 tests, single-line grout pocket connections used #9 bars with a clear spacing of approximately $2d_b$ to $2.5d_b$. Double-line grout pockets used #9 bars with a clear spacing of approximately $9.5d_b$. All connectors were embedded only $13d_b$. i.e., half the cap depth. Splitting cracks did not develop in any test, nor did connectors appear to appreciably slip, even for failure tests in which anchorage was specifically challenged. Connectors will normally be embedded a depth at least 1.7 times that used in Phase 2 and 3 tests (Section 6.3.9). Furthermore, Section 6.3.10 recommends minimum confining reinforcement around all connections. For these reasons, a minimum connector clear spacing of $2d_b$ is considered reasonable and conservative, despite the paucity of test data. For connectors that use a smaller spacing, an increase in connector embedment depth of 50 percent is recommended. This is similar to the penalty imposed by ACI 318-99 for bars with a clear spacing less than $2d_b$ without minimum ties. For reasons of constructability, in no case should the clear spacing be less than d_b or 1 in.

Headed bars are expected to develop anchorage along the bar length much like a straight bar. However, if anchorage along the bar were to be lost due to splitting, the head would engage and could generate a concrete breakout failure. Thus, the clear spacing requirements of $2d_b$ need not be similarly imposed on headed bars. However, there is little design strength advantage in using close spacing for headed bars. As a minimum, a clear spacing of d_b between connector heads should be provided for constructability reasons.

6.3.7.2.2 Duct Spacing for Grouted Vertical Duct and Bolted Connections

Adequate duct spacing should be provided for similar reasons. However, the use of ducts complicates minimum spacing requirements. Based on a likely minimum duct diameter of 3.5 in. and a minimum clearance of 1.5 in. between ducts to ensure proper flow and consolidation of concrete, connector spacing would be a minimum of 5 in. This corresponds to a clear spacing between bars of approximately $2.5d_b$ for #11 bars. However, Phase 1 tests indicated that clear spacing between ducts, not bars, is the more appropriate parameter for design, since the connector-grout unit generated splitting cracks in the surrounding concrete. With clear spacing based on duct diameter, larger connector spacing should be expected.

Phase 2 tests used a clear spacing between ducts of two to three duct diameters. In contrast to the splitting cracks around the ducts observed in Phase 1 pullout tests, Phase 2 tests did not exhibit splitting cracks or evidence of connector slip, even for failure tests. Ducts within the cap and pile in Phase 3 pile bent tests used a clear spacing of one to two duct diameters. For the failure test, connectors on the tension side yielded without the development of any splitting cracks on the cap surface. Piles exhibited only flexural cracks. Although these tests used smaller bars than the Phase 1 tests (#9's in Phases 2 and 3 vs. #11's in Phase 1), the embedment depth was only $13d_b$. It can be reasoned that for embedment depths that will normally be used in a precast bent cap system ($24d_b$ or more for grouted vertical duct and bolted connections, based on Class C concrete and Grade 60 reinforcement), splitting failure will be unlikely even for larger #11 bars, as long as a clear spacing of at least one duct diameter is used.

Based on these test results, a minimum clear spacing between ducts of one duct diameter is recommended when the connector embedment depth is based on development length requirements of Section 6.3.9. Despite the paucity of test data, this is considered reasonable and conservative, as connectors will normally be embedded a depth at least 1.8 times that used in Phase 2 and 3 tests and minimum confining reinforcement will be used around ducts (6.3.10). Although this requires a larger connector spacing than for grout pocket connections, this provision should not place an excessive limitation on grouted vertical duct connections. For example, #11 connectors using 4-in. ducts with a clear spacing of one duct diameter would effectively require a connector clear spacing of approximately $5d_b$. Although this is 2.5 times the required clear spacing for grout pocket connectors, it can still be accommodated reasonably well in design because piles will not normally require many connectors and columns can be sized and shaped to maximize spacing (e.g., larger columns or columns of square or rectangular cross section).

For duct spacing smaller than one duct diameter, an increase in connector embedment depth of 50 percent is recommended for both straight and headed bars. For reasons of constructability, in no case should the clear spacing between ducts in the cap or column be less than 1.5 in. or 4/3 times the maximum coarse aggregate size. Ducts in piles and columns should also maintain a cover of at least 3 in. when embedment depths are determined according to 6.3.9. For smaller cover, a 50 percent penalty on embedment depth applies.

Because bolts extend through the entire cap depth, the previously mentioned duct spacing limitations should not be applied the same to bolted connections. Duct spacing for bolted connections is limited by the minimum requirements of 1.5 in. or 4/3 times the maximum coarse aggregate size. Use of high-strength steel for bolts should not pose a problem for anchorage because of the large embedment depth used and the redundancy provided by the cap top anchorage.

In developing connector arrangements, designers should carefully position the cap and pile ducts, as duct sizes will likely differ. Potential interference with cap and column or pile reinforcement should be considered and avoided. Potential conflicts become more critical as the duct size and number of connectors increase.

6.3.7.3 Minimum Connector Reinforcement

Provision in the AASHTO Standard Specifications for minimum column longitudinal reinforcement of one percent (as a percentage of the column cross-sectional area) is based on limiting the effects of creep and shrinkage under sustained compressive stresses [6.11, Section R10.9.1]. Thus, connector design need not be governed by this limitation. As mentioned previously, behavior of a precast bent cap system is expected to approach (but not equal) that of cast-in-place bents as the percentage and distribution of connector reinforcement approaches the minimum one-percent column reinforcement typically used in Texas. However, if limit states are properly designed for, use of one percent connector reinforcement is not necessary.

The following two provisions for minimum connector reinforcement are recommended to ensure the system possesses adequate redundancy and structural integrity, regardless of potentially smaller connector reinforcement requirements determined from analysis:

1. A minimum percentage of connector reinforcement of 0.7 percent of the gross area of the column and one percent of the gross area of the pile
2. A minimum of four connectors per connection for cast-in-place columns and three connectors for trestle piles. Connectors should be well distributed over the column cross section.

Minimum requirements are not additive to strength design requirements.

In addition, it is recommended that connector size be limited to the following:

1. For standard reinforcing bars: #7 to #11 bars
2. For bolted connections: diameters between 7/8 in. and 1.5 in., inclusive.

The respective recommendations of 0.7-percent and one-percent connector reinforcement for columns and piles is based on engineering judgment, test results, and existing code provisions. Based on excellent performance of Phase 2 and 3 specimens, which used connector reinforcement of 0.45 to 0.57 percent for columns and 1.2 to 1.6 percent for piles, requirements of one percent for columns and 1.5 percent for piles are considered unnecessarily conservative for many applications. In fact, Section 5.7.4.2 of AASHTO LRFD uses a lower bound of 0.7 percent for column longitudinal reinforcement in Seismic Zone 1. Both AASHTO Standard Specifications and AASHTO LRFD require a minimum longitudinal reinforcement for precast piles of 1.5 percent and four bars. Thus, the recommendations for a precast bent cap system require connector reinforcement that is approximately 30 percent less than the longitudinal bars that are typically extended into cast-in-place caps.

As mentioned previously, the 0.7-percent and 1-percent requirements are intended to satisfy the AASHTO requirement for structural integrity by ensuring connections possess an additional capacity to transfer forces beyond that which may be required by governing load combinations. This is important for precast bent cap systems because the designer may calculate a very small amount of required connector reinforcement for the common case of small design eccentricities. Without a minimum reinforcement requirement, structures may possess little reserve to transfer forces associated with unexpected events.

This provision also satisfies the ACI requirement for minimum tensile capacity of precast columns [6.11, Section 16.5, 6.13]. A nominal strength in tension not less than $200A_g$ (in pounds), where A_g is the gross column area in in^2, corresponds to a required percentage of connector reinforcement of 20/f_y (in percent), where f_y is the specified yield strength of the connector in ksi. For connectors with 60-ksi yield strength, a minimum connector reinforcement of 0.33 percent would be required. Thus, a minimum reinforcement requirement of 0.7 percent requires approximately twice as much as this ACI requirement.

The recommendation for a minimum of four connectors for column bents and three connectors for trestle pile bents is intended to provide redundancy. Distribution of the connectors over the cross section ensures that some connectors provide tensile resistance. As few as three connectors is considered suitable for the smaller caps used in trestle pile bents.

Using #8, #9 and #11 connectors as a basis, Table 6.2 shows the expected impact of minimum connector requirements on design. For trestle pile bents, 2/2 configurations are most likely to be used, although other configurations are feasible. For example, a three-connector design is viable for a 16-in. pile if #9 bars (or similar diameter bolts) or larger are used. A 1/3 configuration can be used for an 18-in. pile if a #10 or #11 bar is used. A 2/2 configuration is very suitable for #8 bars or larger. This configuration also provides a large duct spacing, thereby avoiding the requirement of a 50% increase in embedment depth for connectors in grouted vertical duct connections. When connectors are small, achieving the minimum connector percentage will result in an excessive number of connectors, thus leading to congestion, more difficult construction, and thus a less efficient and more expensive design. In most pile bents, bar sizes will likely be limited to #9's or 1-1/8 in. bolts. Embedment depth requirements for larger bars may preclude their use in grout pocket and grouted vertical duct connections for trestle pile bents. A 2/3 configuration would not normally be required, except for larger piles subjected to unusually large eccentricities.

For column bents, at least a 2/2 connector arrangement is required. The minimum connector percentage requirement eliminates 4-#9's, 4-#11's, and even 6-#9's for many columns. For a 30-in. column, 4-#10's or larger are required, and for a 36-in. column, 6-#10's or larger are required. Six #11's provide 0.68% for a 42-in. column, which may be interpreted as nominally acceptable. It is the designer's prerogative to use a smaller percentage of reinforcement for cases in which small design loads and combined eccentricities are expected, although this is not recommended. Phase 2 and 3 tests demonstrated acceptable response at service and factored levels for percentages of 0.57 percent for #9 bars and 0.45 percent for 1-in. bolts. However, minimum provisions are intended to encourage the designer to increase the number and size of connectors to incorporate redundancy and structural integrity. Even if a designer chooses to use a percentage of connector reinforcement smaller than 0.7 percent, a minimum of four connectors should still be used for redundancy.

6.3.7.4 Upper Limit on Connector Reinforcement

Limitations on size and spacing of connectors and ducts will limit connector reinforcement to approximately two percent (see Table 6.2). For constructability, designers should limit connector reinforcement to this limit. Percentages slightly higher may be permissible if constructability is carefully considered.

6.3.8 Analysis of Connector Configuration

6.3.8.1 Introduction

After a trial connector configuration is selected, the connection is analyzed to ensure it satisfies requirements for strength and serviceability limit states. If vehicle collision or vessel impact is not prevented, analysis at the extreme limit state is also required. For the strength limit state, axial load-moment design strength and shear are checked. Confining reinforcement in the joint region is addressed in Section 6.3.10.2. Serviceability checks include: 1) potential opening at the bedding layer, 2) cracks in the connection region at the cap top, and 3) bent deflections. Based on these checks, the connector configuration may require modification. Thus, the design process is iterative. Occasionally, the connection type or bent configuration may also be modified.

6.3.8.2 Strength Limit State

Connection design at the strength limit state requires that an adequate number and configuration of connectors are provided to resist axial load-moment interaction as well as shear. Design for axial load-moment interaction uses the familiar axial load-moment (P-M) design strength curves and equations. Connection design for shear forces requires prevention of two different potential failure mechanisms: 1) direct shear transfer failure at the bedding layer, and 2) joint shear failure.

Connection design at the strength limit state should also provide a suitable margin of safety. To this end, it is recommended that the following two provisions be adopted for connection design at the strength limit state:

1. A 30 percent increase in factored design actions
2. The use of a conservative strength reduction factor, ϕ, as described in subsequent sections.

The 30 percent increase in factored design actions provides an additional safety margin to indirectly provide for connection ductility, as discussed in Section 6.3.1.2. This is in addition to the conservatism included in design actions by assuming rigid connections in analysis.

The purposes of a strength reduction factor, ϕ, are [6.11, Section R9.3.1]: 1) to account for inaccuracies in design equations, 2) to reflect the degree of ductility and required reliability of the connection under load effects, 3) to reflect the importance of connections in a precast bent cap system, and 4) to allow for the probability of understrength connections due to variations in material strengths and dimensions. Subsequent sections incorporate different values of ϕ, based on the four factors mentioned above, as well as the level of conservatism used in deriving design equations from limited test data. Even in cases in which a ϕ factor of 0.9 is used, considerable conservatism is incorporated in design equations.

6.3.8.2.1 Axial Load-Moment Interaction

The capacity of the trial connection configuration is first checked by comparing the factored axial load–moment combinations determined from analysis to the axial load-moment design strength curves. Factored axial load–moment combinations are determined from analysis as explained in Section 6.3.5. Appropriate approaches, such as the Load Contour Method or the Reciprocal Load Method, should be used to account for biaxial bending. Reference 6.15 provides a brief summary of these approaches and conditions for which they apply.

Because TxDOT typically uses standard column and pile sizes and because there are a limited number of practical connection arrangements, the designer may desire to generate a family of axial load-moment design strength curves to facilitate design. Figures 6.10 and 6.11 show example design strength curves (uniaxial bending) for longitudinal and transverse bending of an 18-in. pile and 30-in. column, based on Figures 6.8 and 6.9, respectively. Grade 60 steel is assumed for connectors.

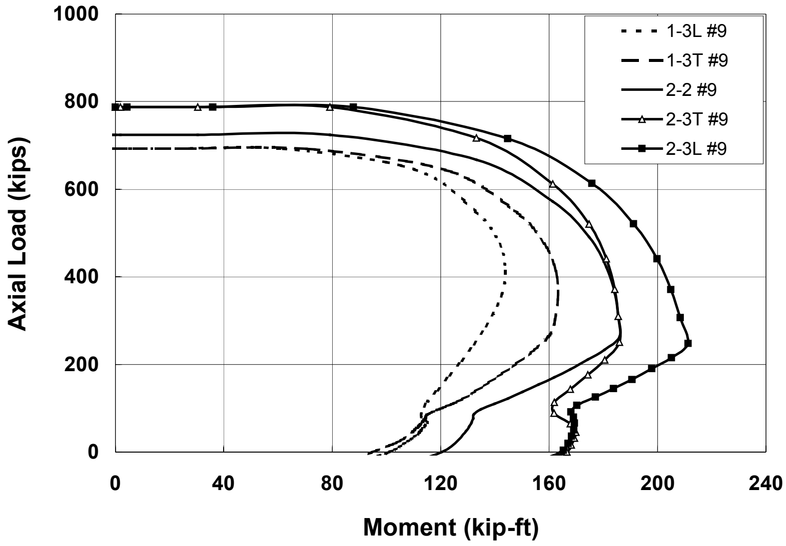


Figure 6.10 Axial Load-Moment Design Strength Interaction Curve—18-in. Pile

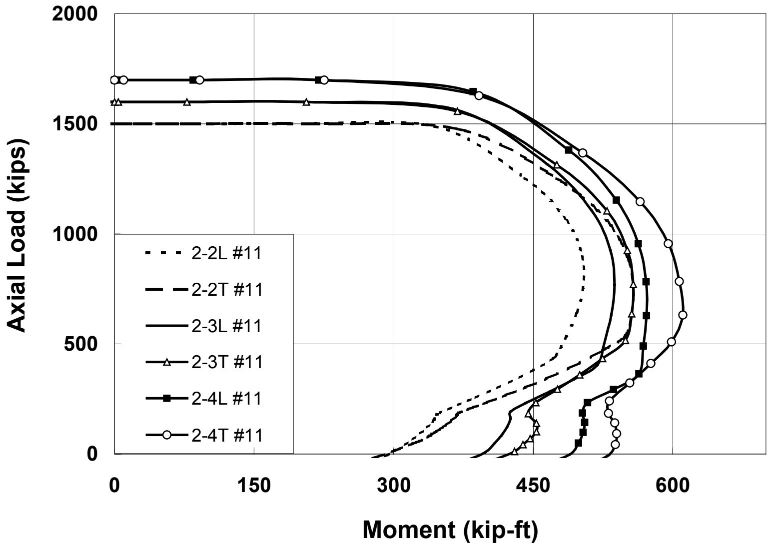


Figure 6.11 Axial Load- Moment Design Strength Interaction Curve—30-in. Column

6.3.8.2.2 Shear Friction

For surface-flush connections, resistance to direct shear at the boundary between the bent cap and column or pile top may be checked using the shear friction model specified in AASHTO LRFD [6.10, Section 5.8.4]. AASHTO LRFD assumes relative displacement is resisted by cohesion and friction, with friction being maintained by the shear friction reinforcement crossing the crack. This provision is similar to those found in the AASHTO Standard Specifications and other references [6.11, Section 11.7.4, 6.14, Section 4.3.6]. The recommended approach for a precast bent cap system is the LRFD approach. For a precast bent cap system this approach takes the nominal shear resistance of the interface plane, V_n, as:

$$V_n = cA_{cv} + \mu(A_{vf}f_y + P_c) \leq (0.2f'_cA_{cv},\ 0.8A_{cv}) \qquad (6\text{-}1)$$

where: V_n = nominal shear resistance, kips

A_{cv} = area of concrete engaged in shear transfer, in^2

A_{vf} = area of connector reinforcement crossing the bedding layer, in^2

f_y = specified yield strength of reinforcement, ksi

c = cohesion factor, ksi, equal to 0.075 for a precast bent cap system (concrete and grout placed against hardened concrete not intentionally roughened)

μ = friction factor, equal to 0.6 for a precast bent cap system (normal-weight concrete and grout placed against hardened concrete not intentionally roughened)

P_c = permanent net compressive force normal to the shear plane, kips

f'_c = smaller of the specified 28-day compressive strength of the grout and concrete, ksi

If the bedding layer is intentionally roughened to an amplitude of ¼ in., then a cohesion factor of 0.100 may be used together with a friction factor of 1.0. The upper limits prevent the use of unconservative values for shear resistance. Because the design equation for shear friction is based on well-documented research, a strength reduction factor, ϕ, of 0.85 is recommended for this strength check. The factored shear at the column or pile top may be calculated using a square-root-sum-of-the-squares (SRSS) approach to combine simultaneous shear in the transverse and longitudinal directions due to wind, braking forces, etc.

It has been shown experimentally that when a moment acts on a shear plane, there is no change in the resultant compression acting across the shear plane. Thus, Section 11.7.7 of Reference 6.11 indicates that additional reinforcement is not required to resist the flexural tension stresses. This means that connectors provided for flexural resistance or redundancy may also be used to provide shear-transfer strength. However, if a net tension force exists at the bedding layer, reinforcement used to carry such tension should not be used for shear transfer. On the other hand, if a permanent net compressive force, P_c, exists (e.g., reliable dead load), then the designer may take advantage of its contribution to shear transfer, as provided for in the equation above. In most cases it is expected that the minimum number of required connectors will provide adequate resistance to direct shear. For embedded connections, a direct shear failure is not anticipated. Therefore, design for shear friction is not required.

6.3.8.2.3 Joint Shear

Joint shear should also be considered in transfer of lateral and gravity loads through connections. Although joint shear failure is not expected to govern design, designers should still ensure adequate joint shear strength is provided.

Figure 6.12 portrays the basic force transfer mechanism for a beam-column joint in a cast-in-place frame subjected to gravity and lateral loads. It is recommended that design of beam-column joints in a precast bent cap system follows applicable ACI-ASCE Committee 352 provisions [6.16]. ACI 352 defines joints in a continuous moment-resisting structure designed on the basis of strength without special ductility requirements as Type 1 joints. This includes typical frames designed to resist gravity and normal wind loads, but not earthquake-induced forces.

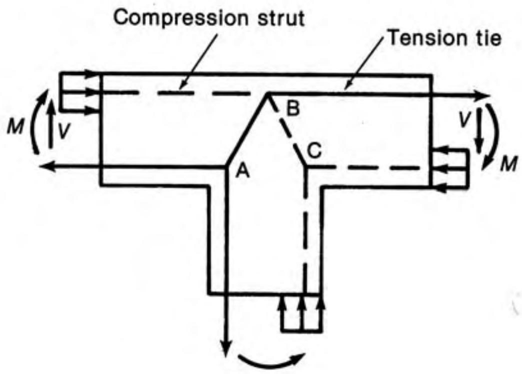


Figure 6.12 Beam-Column Joint Forces Based on Truss Model [6.26]

ACI 352 requires that the joint shear strength exceed the maximum joint shear on a horizontal plane at the joint mid-height. This ensures that the strength of a compressive strut is sufficient to resist the applied loads. Designers may alternatively develop a strut and tie model to check joint shear [6.10, Section 5.6.3]. Joint shear on a horizontal plane at mid-height of the joint is limited to the nominal joint shear strength, V_n, times a strength reduction factor, ϕ, of 0.85, where V_n is determined as follows:

$$V_n = 12\sqrt{f'_c}\ b_j h \qquad (6\text{-}2)$$

where: V_n = nominal shear strength, lbs.

f'_c = concrete compressive strength, psi, limited to 6000 psi

b_j = effective joint width, in.

h = thickness of the column in the direction of load being considered, in.

AASHTO Standard Specifications mentions that the strength of column connections in a column cap is relatively insensitive to the amount of transverse reinforcement, provided there is a minimum amount and that a limiting shear stress of $12\sqrt{f'_c}$ is used. Confinement reinforcement is addressed in Section 6.3.10.

The joint shear is determined from a free-body diagram of the joint and is based on the development of the flexural strength of cap or column at the connection rather than factored loads. In cases where the column (i.e., connector) flexural capacity is less than that of the beam, the column flexural capacity should be used to determine connection forces. Flexural steel stresses should be taken as αf_s, where f_s is the specified yield stress of the longitudinal reinforcement and $\alpha \geq 1.0$. Designers may use a value of α larger than 1.0 to account for the actual yield stress being larger than the nominal value.

The designer should also consider placement of the top longitudinal reinforcement with respect to the column or pile cross section. The diagonal compression at the lower end of the strut is vertically equilibrated by the column thrust, as shown in Figure 6.12. Thus, if the engineer is compelled to design the top longitudinal reinforcement outside the column or pile cross section (in plan view), then vertical stirrups should be provided to resist the vertical component of the thrust from the compression strut [6.15, Section 10.2].

6.3.8.3 Service Limit State

Design for serviceability includes the following checks: 1) potential opening at the bedding layer, 2) cracking in the connection region at the cap top, and 3) deflections of the bent.

6.3.8.3.1 Opening at the Bedding Layer

For cases in which the bedding layer is exposed and uncoated connectors are used, opening at the bedding layer under service loads may expose connectors to moisture, chlorides or other aggressive agents, possibly leading to deterioration of the connection. This is an issue for a precast bent cap system because of the likely use of a relatively small number of connectors (compared to cast-in-place bents) and the location of such connectors within the core of the column or pile. Prediction of actual openings is difficult. However, the likelihood of any opening may be conservatively estimated by determining the location and orientation of the neutral axis for service-level load combinations. Existing software such as PCACOL or UCFyber [6.6,6.17] may be used to conduct section analysis. In conducting an analysis, the following should be observed:

1. Frame analyses may conservatively assume rigid connections
2. Service-level gravity and wind loads used in the analysis should be carefully determined
3. Simultaneous bending moments in the transverse and longitudinal directions should be considered
4. Connector reinforcement, not column bars, should be used in defining the section for analysis.

If the section analysis indicates that the neutral axis is located such that one or more connectors is subjected to tension, opening might be of concern. Phase 2 and 3 tests indicated that such an analysis should be regarded as a conservative and approximate means of determining a potential opening. In Phase 2 tests, tension was produced in connectors for Tests 2 through 4, similar to the pattern shown in the analysis of Figure 4.57. However, only the use of a very large eccentricity (55.5 in.) for Test 4 actually produced a measurable opening of more than 0.002 in. During Phase 3 tests, an opening of 0.003 in. opened during the second proof test of the column bent (CB2), in which transverse and longitudinal eccentricity combinations at column tops were as large as 2.7 in. and 1.6 in, respectively. Analysis confirmed the expectation of an opening, although the neutral axis was not predicted to have repositioned such that a connector would be in tension. As for Phase 2, excessive openings developed upon the application of the large eccentricities.

When evaluating possible openings, the engineer should also consider the likely duration. For example, potential openings due to unbalanced gravity loads in a coastal region, where coastal spray may penetrate an opening, may be viewed more seriously than a transient opening due to wind loading.

Engineering judgment is required to decide what course of action to take if the analysis indicates a potential opening. In cases where durability protection is of primary importance, one or more of the following measures may be taken:

1. *Embed the column or pile into the cap.* This approach was used for select Phase 2 and 3 connections and successfully prevented openings. Embedment is highly reliable and not expected to significantly impact construction cost, although it can obstruct visibility of shims during cap setting operations and limit post-grouting inspection.
2. *Use epoxy-coated connectors.* This approach may be appropriate for cases where minor opening is possible, but surface-flush connections are preferred. The use of epoxy-coated connectors are viable for all cases except bolted connections. Other alternatives may be available for bolts such as galvanized steel. The cost impact of epoxy coating is expected to be minor. Development length provisions already account for epoxy coating, as all tests were conducted with epoxy-coated bars.
3. *Use an external sealant.* A sealant can prevent moisture penetration at the bedding layer and can be inexpensive and easy to apply. However, inspection of sealants may be difficult in some cases. An external sealant may also be applied to exposed grout surfaces at the cap top.

4. *Use other measures.* Other measures such as a water stop or post-tensioning may be viable. A water stop would be relatively expensive. Post-tensioning may be a simple and inexpensive option for bolted connections.

6.3.8.3.2 Cracking at the Top of the Bent Cap

All of the Phase 2 and 3 specimens exhibited cracks at the cap top in the connection region under full service-level loads. Such cracks may threaten the durability of a connection due to their size and location. Cracks are of concern in two different locations: 1) grout pockets, and 2) the surrounding concrete.

Phase 2 and 3 specimens exhibited significant cracking within grout pockets, but negligible cracking in ducts that were grouted. Cracks as wide as 0.009 in. appeared at the surface of the Phase 2 single-line and double-line grout pockets at service-level loads during Test 1. Although subsequent tests showed cracks as large as 0.016 in. at service level, this increase in crack width is attributed to the application of factored loads during previous tests. Flexural cracks in the concrete were arrested by ducts used in vertical duct and bolted connection tests. Cracks did not appear at the grout surface of any ducts. During Phase 3 tests, no cracks developed at the surface of the bent cap in either the concrete or grout. For column bent tests, crack widths were limited to 0.007 in. within double-line grout pockets at factored loads. No cracking developed within grouted ducts. No clear correlation between crack width and grout types or strengths was evident at service level in Phase 1-3 tests.

In the connection vicinity, flexural crack widths within the concrete were typically of the same magnitude as the maximum grout pocket cracks: 0.009 in. at service level and as large as 0.016 in. during subsequent tests. Shear cracks on the sides of the cap exhibited widths that were considerably smaller.

Current provisions in the AASHTO Standard Specifications adopt ACI 318-95 provisions, which used the empirical Gergely-Lutz equation and a maximum permissible crack width of 0.0016 in. for interior exposure and 0.013 in. for exterior exposure [6.3]. The statistically-derived Gergely-Lutz equation predicts the maximum crack width, w, from the following:

$$w = 0.076\beta f_s \sqrt[3]{d_c A} \qquad (6\text{-}3)$$

where: w = maximum crack width, 0.001 in.

$ß$ = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement

f_s = calculated stress in the tension reinforcement at service load, ksi

d_c = thickness of cover measured from extreme tension fiber to center of closest longitudinal reinforcing bar, in.

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, in^2

For design, this expression is often formatted in terms of an allowable reinforcement stress, f_s, or a crack control parameter, Z, defined as follows:

$$Z = f_s \sqrt[3]{d_c A} \qquad (6\text{-}4)$$

where: Z = crack width parameter, kips/in.; limited to 170 for an allowable crack width of 0.016 in. and 140 for a crack width of 0.013 in.

Predicted crack widths using Equation 6-3 were approximately twice as large as the observed crack widths in tests. For example, for the Phase 2 PSL1 test, a maximum service-level flexural crack width of approximately 0.009 in. was predicted for the specimen, whereas a maximum crack width of 0.005 in.

was measured. The maximum predicted service-level crack width for the first test of CDL, CVD, and CBC specimens was 0.014 in., compared to the measured crack width of 0.007 in. Given the relatively large inherent scatter in crack width data, predictions using Equation 6-3 were reasonable and conservative. This is not surprising, given that flexural cracks for a precast bent cap may develop similar to a cast-in-place cap. Hence, existing AASHTO provisions for crack widths are considered useful.

In the 1999 edition of ACI 318, Equation 6-3 was eliminated in favor of a limitation on the maximum longitudinal reinforcement spacing, s [6.11, Section 10.6.4]:

$$s = \frac{540}{f_s} - 2.5c_c \le 12\left(\frac{36}{f_s}\right) \tag{6-5}$$

where: s = center-to-center spacing of flexural reinforcement nearest to the extreme tension face, in.

f_s = calculated stress in the tension reinforcement at service load, ksi

c_c = clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement, in.

Equation 6-5 is more designer-friendly and is intended to control surface cracks to a width that is generally acceptable in practice, but that may vary widely in a given structure.

It is important to note that this new provision is not sufficient for structures subject to very aggressive exposure [6.11, Section 10.6.5]. ACI Committee 224 has recommended that cracks be limited to 0.007 in. for structures exposed to deicing chemicals and 0.006 in. for exposure to seawater and seawater spray [6.18].

Provisions for a precast bent cap system should ensure that the connection region provides protection at least equal to that of the surrounding concrete. The following provisions are, therefore, recommended for crack control of a precast bent cap system:

1. For grout pocket connections, estimate the maximum crack width in the grout pocket using Equation 6-3 and assuming a maximum allowable crack width of 0.013 in. for normal exposure conditions. Equation 6-4 with an allowable crack width parameter, Z, of 130 kips/in may also be used. Smaller tolerable widths should be used for aggressive exposure conditions. Although provisions of the AASHTO Standard Specifications may change in the future to correspond to the new ACI 319-99 provisions, it is recommended that the check of estimated crack width not be eliminated, as some researchers feel that the new ACI provision may not be as widely applicable and conservative as previous crack control provisions.
2. For all connection types, limit the spacing of longitudinal reinforcement in the cap to the maximum permissible spacing according to Equation 6-5. Spacing limitations may tend to govern especially for grout pocket connections that use large pocket widths.
3. For cases in which crack widths in grout pockets are expected to be excessive or for aggressive environments, an external sealant may be applied to exposed surfaces, grout pocket heights may be reduced to less than full-depth, or the connection type may be changed to a grouted vertical duct or bolted connection. The designer should not attempt to reduce the parameter, d_c, in Equation 6-3 in an attempt to reduce crack width, and should use a value of d_c equal to 2.0 in. in Equation 6-3 for cases in which d_c is calculated to be greater than 2.0 in. For protection of reinforcement in the surrounding concrete, typical measures may be taken to reduce the expected crack width, such as using a larger number of smaller diameter bars or reducing the steel stress.

It is important for the designer to remember that, although crack width is important, concrete quality, protective coatings, minimum cover, distribution and size of reinforcement, and reinforcement details should also be considered in design [6.10, Section 5.12]. To enhance durability protection, AASHTO LRFD requires adequate cover, nonreactive aggregate-cement combinations, thorough consolidation of the concrete, adequate cement content, low water/cement ratio and thorough curing, and allows for the use of protective coatings over reinforcement such as epoxy coating and galvanization.

6.3.8.3.3 Deflections

To estimate deflections, the designer should conduct two analyses, one assuming the bent cap-to-column (or pile) connections are pinned, and the other assuming rigid connections. Engineering judgment is required to estimate realistic response. Analyses for cast-in-place column bents typically assume rigid joints, often based on the extension of one-percent longitudinal column reinforcement into the cap. Figure 6.5 showed how the number and position of connector steel affected the section stiffness. If the conservative assumption of a pinned connection in analysis results in excessive bent deflections, then it is recommended that the engineer estimate deflections using a reasonable estimate of connection stiffness. This may be based on the relative connection stiffness for the connector configuration and a cast-in-place column using one percent longitudinal reinforcement distributed around the perimeter (Figure 6.3).

If deflections are potentially problematic, the designer may enhance rotational stiffness of the connection by: 1) increasing the number and area of connectors, or 2) increasing the stiffness of the columns (or piles) and/or bent cap. The effect of embedding the column or pile into the cap may enhance connection stiffness, but insufficient data is available to quantify this effect.

6.3.9 Determination of Connector Type and Embedment Depth

6.3.9.1 Introduction

After the selected connection configuration is analyzed for strength and serviceability, the connector type and embedment depth are determined. For grout pocket and grouted vertical duct connections, selection of connector type requires decisions regarding: 1) anchorage—straight or headed bars, and 2) connector coating—black or epoxy-coated bars. Based on these decisions, the connector embedment depths into the cap and column or pile are determined. Equations for both straight and headed connectors are provided. For bolted connections, the connector type may be selected from various alternatives.

6.3.9.2 Connector Type

6.3.9.2.1 Anchorage

Use of straight bars provides many advantages, such as being inexpensive, readily available, non-proprietary, and having well-defined properties. Straight bars also accommodate horizontal tolerances more easily than headed bars. Prior to testing, the primary disadvantage of straight bars was thought to be the potentially large development lengths. However, Phase 1-3 tests of grout pockets and grouted vertical ducts indicated that straight, ASTM A615, Grade 60 reinforcing bars provide adequate anchorage for the common range of bar sizes and bent cap depths. Confinement effects related to cover, typical cap reinforcement and stresses in the connection region helped bars achieve up to 1.2 times the yield strength at embedment depths of approximately $13d_b$ for grout pocket and grouted vertical duct connections. Little distress was observed in the connection region in Phases 2 and 3. Thus, straight bars provide a viable alternative for precast connections.

It is recommended that when connectors are used with grout pockets or grouted vertical ducts the diameter is limited to that of a #11 reinforcing bar and a specified yield stress of 75 ksi because the semi-empirical equations derived in subsequent sections are based on bar sizes and strengths in this range. This is in accordance with standard practice for typical bridge substructures. Use of higher strength bolts is acceptable, as both bond and bearing at the cap top anchorage are available to transfer tension from bolts.

Because of the fairly short development lengths required for straight bars used in grout pockets and grouted vertical ducts, headed bars will not normally be required. However, there are some cases for which the designer may choose to use headed bars, including: 1) bents for which additional redundancy (i.e., reserve capacity) is desired, 2) bents subjected to dynamic or unusually large lateral forces, and 3) trestle pile caps for which an insufficient depth is available for straight bar anchorage in grout pockets.

The use of headed bars provides the designer a convenient approach to build additional redundancy or reserve capacity into the system without increasing the number of connectors. Phase 1 testing of headed bars indicated that if the embedment exceeds that required for development, the bearing force at the head may be limited and thus the head may play a small role in force transfer. In such a case, which would correspond to normal practice, headed bars provide an additional load transfer mechanism if bond is degraded or unexpectedly lost. Thus, headed bars provide redundancy and therefore additional conservatism. Some cases for which this may be desirable include shallow bent caps, large-diameter and/or high-strength connectors, close connector spacing, or questionable grouting operations or grout strength.

The use of headed bars may also be desirable for cases in which dynamic or unusually large lateral forces are possible. While a precast bent cap system, as developed herein, is not intended to resist seismic loading, there may be cases in which moderate dynamic loading, such as coastal winds, is possible. In such cases, the designer may prefer to use headed bars to enhance the integrity of the lateral force resisting system, in the event that bond along the connector degrades under repeated loading. Given the limited scope of testing and lack of reversed cyclic loading during tests, the engineer must use judgment to decide if the use of a precast bent cap system is deemed appropriate for a particular bent configuration and specific characteristics of dynamic loading. As mentioned in Chapter 1, it has been found that there is a minimal reduction in capacity and minimal head slip for tension cyclic loading of headed bars for loading up to 80 percent of the elastic range (15 cycles with bars debonded along the length), and that high-cycle fatigue loading has no effect on anchor strength when anchors (both cast-in-place and grouted threaded rods) are embedded sufficiently to develop full tensile capacity of the anchor steel under static loads [1.52,1.53]. The designer should be cautioned that these were limited studies.

The designer must also decide on the type of proprietary headed bar. Testing during Phases 1-3 was limited to the use of the HRC upset-headed bar. A prime advantage of the upset-headed bar for a precast bent cap system is its small head diameter of approximately $1.4d_b$ (i.e., a head area twice that of the bar) which readily accommodates tight horizontal tolerances. As mentioned in Section 2.3.2, anchorage performance should be between that of straight bars and the larger HRC T-headed bars or the Lenton *Terminator*, both of which use various head shapes with a head diameter at least twice that of the upset head. Based on Phase 1-3 tests, any of these headed bars are expected to provide adequate anchorage. The designer should be particularly careful, however, in choosing a headed bar that accommodates the necessary construction tolerances. Costs and field construction approaches also vary widely for the different headed bars and should also be considered in the selection process.

If a bolted connection is selected, a large variety of proprietary and nonproprietary threaded bolts are available from manufacturers such as Dywidag and Williams (Section 2.3.5). Factors affecting the designer's selection of bolts may include: ease of constructability related to cap setting, cap top bearing, and/or post-tensioning, bolt tensile strength, and availability and cost.

Sections 2.3.3 and 2.3.4 introduced hooked bars and U-shaped bars. The designer should consider constructability and durability when selecting these options. For example, hooked bars and U-shaped bars are feasible only in grout pocket connections, due to the space required for 90-degree or 180-degree hooks. For similar reasons, the total number of connectors is also limited. In addition, if required, epoxy coating must be applied after connectors are bent. It should also be recognized that U-shaped bars require embedment at both ends of the connector.

6.3.9.2.2 Coating

In selecting the connector type, the designer must also decide if epoxy coating should be used. All tests in Phases 1-3 were conducted using epoxy-coated connectors to ensure the influence of epoxy coating on anchorage is conservatively accounted for in design. Tests revealed that short development lengths result even for epoxy-coated straight bars. Thus, the designer is encouraged to use epoxy-coated bars whenever necessary for durability enhancement. When bolted connections are selected, the designer should check with manufacturers for alternative corrosion protection measures.

6.3.9.3 Embedment Depth

Because steel failure is typically more predictable and ductile than a concrete failure, the preferable failure mode for connectors in a precast bent cap system is bar yield followed by fracture, rather than concrete breakout, bar pullout or splitting. To ensure this failure mode, an adequate embedment depth must be provided. The required embedment depth depends on both the connector anchorage as well as the connection type. The following sections define conservative design provisions for embedment depth, which are based on data analyses provided in Section 6.4. Provisions found in Reference 6.10 or 6.11 may be used for hooked and U-shaped bars.

6.3.9.3.1 Straight Bars in Grout Pockets

The required embedment depth for straight reinforcing bars embedded in grout pocket connections may be determined from the following equation:

$$l_d = \frac{0.022 d_b f_y}{\sqrt{f'_c}} \qquad (6\text{-}6)$$

where: l_d = development length, in.

d_b = nominal diameter of bar, in.

f_y = specified yield strength of connector, psi

f'_c = specified concrete compressive strength of the bent cap, psi

As shown in Section 6.4.2.3.1, Equation 6-6 is based on Phase 1 test data, as well as Phase 2-3 test results. A safety factor of approximately 1.7 has been included by accounting for strain hardening and yield stresses larger than specified, discounting a portion of the available strength, and including a strength reduction factor. This reflects the importance of connection reliability and ductility and the paucity of data on which Equation 6-6 is based.

Since all tests were conducted on epoxy-coated bars, this equation may be conservatively applied to both coated and uncoated connectors. For the common case of a Grade 60 reinforcing bar with f'_c of 3600 psi, the required development length is $22d_b$, or approximately 1.7 times that used to develop connectors in Phases 1-3.

6.3.9.3.2 Straight Bars in Grouted Vertical Ducts

The required embedment depth for straight reinforcing bars embedded in grouted vertical duct connections may be determined from the following equation:

$$l_d = \frac{0.024 d_b f_y}{\sqrt{f'_c}} \qquad (6\text{-}7)$$

where: l_d = development length, in.

d_b = nominal diameter of bar, in.

f_y = specified yield strength of connector, psi

f'_c = specified concrete compressive strength of the bent cap, psi

Similar to Equation 6-6, Equation 6-7 is based on Phases 1-3 test results (see Section 6.4.2.3.2), and incorporates a safety factor of approximately 1.7. This equation may be conservatively applied to both coated and uncoated connectors. For the common case of a Grade 60 reinforcing bar with f'_c of 3600 psi, the required development length would be $24d_b$, approximately 10 percent longer than that required for straight bar anchorage in grout pockets. This required length is 1.8 times that used to develop connectors in Phases 1-3.

6.3.9.3.3 Headed Bars in Grouted Vertical Ducts

As discussed in Section 6.4.3, Equation 6-7 is conservatively applied to headed reinforcing bars in grouted vertical ducts.

6.3.9.3.4 Headed Bars in Grout Pockets

The required embedment depth for headed reinforcing bars used in a grout pocket connection may be determined using a modified Concrete Capacity Design (CCD) equation for concrete breakout strength (Section 6.4.4.1). The nominal concrete breakout strength for a group of connectors in tension, P_{cbg}, may be determined from the following equation:

$$P_{cbg}=\frac{A_N}{A_{No}}\Psi_E\Psi_C P_b \qquad (6\text{-}8a)$$

where: P_{cbg} = nominal concrete breakout strength in tension of a group of fasteners, lbs

A_N = projected concrete failure area of a fastener or group of fasteners, in^2, not to exceed nA_{No}, where n is the number of tensioned fasteners in the group

A_{No} = projected concrete failure area of one fastener, when not limited by edge distance or spacing, in^2
= $9h_{ef}^2$

Ψ_E = modification factor to account for edge distances smaller than $1.5h_{ef}$

= 1 if $c_{min} \geq 1.5h_{ef}$

= $0.7+0.3\dfrac{c_{min}}{1.5h_{ef}}$ if $c_{min} < h_{ef}$

Ψ_C = modification factor to account for grout pocket connection cracking

= 0.75

P_b = basic concrete breakout strength in tension of a single fastener, lbs

= $24\sqrt{f'_c}h_{ef}^{1.5}$ $h_{ef} \leq 11$ in. (6-8b)

= $16\sqrt{f'_c}h_{ef}^{5/3}$ $h_{ef} > 11$ in. (6-8c)

h_{ef} = embedment depth, in.

c_{min} = smallest of the edge distances that are less than or equal to $1.5h_{ef}$, in.

The design coefficients of 24 and 16 used in Equations 6-8b and 6-8c are based on the large database from which the CCD method was developed and correspond to the use of a nominal strength based on the 5 percent fractile, i.e., a 90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength. In effect, this incorporates a safety factor of approximately 1.7 into Equations 6-8b and 6-8c. Group behavior is directly accounted for by the A_N/A_{No} term. Unlike development length provisions, Equations 6-8b and 6-8c are independent of the connector diameter. However, the embedment depth does depend on the cross-sectional area of the bar since the tensile capacity of the connector group is a function of the bar area. As mentioned in the next section, the embedment depth for a headed bar need not exceed the development length requirement for a straight bar as determined from Equation 6-6.

To ensure connector yield is the governing failure mode, the designer should use an embedment depth sufficient to prevent a concrete breakout failure prior to bar yield. For the strength limit state, the strength reduction factor, ϕ, times P_{cbg}, must exceed the maximum force, P_u, associated with the connector group. P_u is determined as follows:

$$P_u=nA_s(1.25f_y)$$

where: P_u = maximum force of connector group, assuming connector yield, lbs

n = number of fasteners in tension in the group

A_s = cross-sectional area of connector, in^2

f_y = specified yield strength of connector, psi

The 1.25 factor accounts for potential overstrength of the connector material in tension. The strength reduction factor, ϕ, may be taken equal to 0.9 for the governing failure mode. A spreadsheet can easily be developed to iteratively determine the capacity of the connector group for various embedment depths.

6.3.9.3.5 Comparisons

Table 6.3 shows development length requirements for a straight bar in a grout pocket and a straight or headed bar in a grouted vertical duct. The required development length depends on bar yield strength, bar diameter, and the square root of the concrete compressive strength. Bars are assumed to be Grade 60 and development lengths apply to both uncoated and epoxy-coated bars. Table 6.3 shows that embedment depth requirements are not excessive for typical bent cap depths. For example, assuming Class C concrete ($f'c$=3600 psi) is used, an embedment depth of 25 in. is required for a straight #9 bar in a grout pocket. For a trestle pile bent with the pile embedded three inches into the cap and 3-in. cover at the top of the bar, a cap depth of 31 in. is required. For 5000-psi concrete, the required cap depth reduces four inches to 27 in. Development lengths increase two inches (i.e., 10 percent) for straight or headed bars in a vertical duct. These required cap depths are in the range of those currently used for cast-in-place pile caps. Cap depths for cast-in-place column bents can exceed pile cap depths by 50 percent or more. It is evident that even for #11 bars sufficient cap depth will normally be available for connector anchorage. If a designer desires a shallower cap depth, a larger number of smaller diameter bars or a higher strength concrete can be used.

Table 6.3 Development Length Requirements (in.) for Straight and/or Headed Bars in Grout Pocket and Grouted Vertical Duct Connections

	Grout Pocket Straight[A]			Vertical Duct Straight/Headed[B]		
bar	f'c (psi)			f'c (psi)		
size	3600	5000	6000	3600	5000	6000
#8	22	19	17	24	20	19
#9	25	21	19	27	23	21
#10	25	21	19	30	26	24
#11	31	26	24	34	29	26

Footnotes

A. $l_d=0.022 f_y d_b/\sqrt{f'_c}$

B. $l_d=0.024 f_y d_b/\sqrt{f'_c}$

Headed bars in vertical ducts require the same embedment as for straight bars. However, headed bars in a grout pocket require use of the modified CCD equation. Although slightly more complex than a development length requirement, this approach requires few calculations and, with the aid of a simple spreadsheet, can be carried out quickly. A direct comparison of required embedment depths for straight and headed bars in grout pockets is difficult due to the many variables.

Designers can use development length equations in early stages of design to help establish the minimum required cap depth for different bar sizes and connection types. In lieu of CCD calculations, the designer may use the straight bar development length requirement of Equation 6-6 for grout pockets as an upper bound for the headed bar development length.

6.3.9.3.6 Straight or Headed Bars in Columns or Piles

The designer must also determine the required embedment depth of connectors in the column or pile. Because the available development lengths into columns or piles will typically be large, no significant challenge exists for anchorage into columns or piles. When cementitious grout is used for anchorage, tension lap splice provisions from Section 12.5.1 of ACI 318-99 are recommended. Class B splices, requiring 1.3 times the basic development length, l_d, should be assumed because the splice unavoidably occurs at the point of maximum stress. The basic development length, l_d, is based on Section 12.2 provisions of ACI 318-99, without an excess reinforcement reduction factor. As discussed in the next section, the specified grout compressive strength should exceed the concrete strength by 1000 psi. Duct and connector spacing should satisfy provisions of Section 6.3.7.2. When epoxy, polyester, or other grouts are used, manufacturer's recommendations should be followed.

6.3.9.3.7 Guidelines for Application of Development Length Requirements for Straight Bars

The following guidelines are recommended for the application of straight bar development length equations, Equations 6-6 and 6-7, to design:

1. *Concrete strength*: Because of limited test data, the specified 28-day concrete compressive strength, f'_c, used in Equations 6-6 and 6-7 should not exceed 6000 psi. TxDOT Class C concrete for typical substructures has a specified 28-day compressive strength of 3600 psi. Although larger compressive strengths tend to enhance anchorage, a potential reduction in embedment depth beyond that corresponding to a compressive strength of 6000 psi should not be used without additional test data. The average f'_c for straight bar tests in Phase 1 was approximately 5500 psi.
2. *Grout Strength*: Grout provisions specified in Chapter 7 should be followed, including a 28-day grout cube strength of at least 5800 psi. In addition, to prevent the grout from becoming a weak link in the connection, it is recommended that the modified grout cube strength exceed the

concrete compressive strength by at least 1000 psi (See Section 7.2.1). Many prepackaged grouts can achieve such strength.

3. *Modification factors*: In most cases, the use of additional modification factors to account for epoxy-coating, reinforcement size, reinforcement location, lightweight aggregate, transverse reinforcement, and excess reinforcement is not recommended. These modification factors are found in Section 12.2 of ACI 318-99 and are adopted similarly by AASHTO Standard Specifications. There are some cases, however, where the designer should use judgment in applying a modification factor.

    a. *Epoxy coating*: All tests were conducted on epoxy-coated bars. Therefore, the influence of epoxy coating on connector anchorage is directly accounted for in design equations. Thus, development length requirements for uncoated bars possess a larger safety factor. Because a comparison of anchorage behavior for epoxy-coated and uncoated bars was not directly investigated, it is recommended that designers not attempt to reduce development length requirements for uncoated bars. Reference 6.21 found no significant effect of epoxy coating on the bond strength of grouted reinforcement.

    b. *Reinforcement size*: As mentioned in Section 6.3.7.3, it is recommended that the range of connector sizes be between #7 and #11 reinforcing bars, inclusive. Straight bar tests included #6, #8, and #11 reinforcing bars. Limited test results do not provide a sufficient basis for the use of a bar size factor.

    c. *Reinforcement location*: When connectors are embedded vertically, or nearly so, no “top bar” factor is recommended. However, if the bar orientation approaches horizontal, the designer may choose to use a factor as large as the 1.3 factor specified in Section 12.2.4 of ACI 318-99 for horizontal bars. All tests used connectors oriented vertically. Some grouts exhibited segregation in grouted connections. Although this might have reduced grout strength near the top of the connections, anchorage forces were smallest there. On the other hand, denser grout that settled at the bottom of connections probably enhanced anchorage.

    d. *Transverse reinforcement*: All tests included typical cap longitudinal reinforcement in the connection region. However, no reduction in development length is recommended, even if minimum confinement reinforcement is provided per Section 6.3.10.2. In contrast to Phase 1 tests, Phase 2 and 3 tests indicated little or no cracking in the connection region, suggesting the contribution of such reinforcement may be minimal.

    e. *Excess reinforcement*: The design philosophy for anchorage of connectors is to ensure development of 1.25 times the specified yield strength of the connector. The development length should not be reduced if a greater amount of connector reinforcement is used than required by analysis.

4. *Minimum embedment depth*: Cap depths for column bents should readily accommodate development length requirements. To encourage the designer to take advantage of available embedment, it is recommended that the required embedment depth be the larger of Equation 6-6 (or Equation 6-7, whichever applies), ¾ of the cap depth, or 18 in. For example, based on Equation 6-7, a #9 bar grouted in a vertical duct would require a 27-in. embedment into the cap, assuming Class C concrete. This is approximately 2/3 of the cap depth for a 42-in. cap. Using the minimum depth requirements, the designer would determine the connector embedment as the largest of {27 in, 0.75(42 in.)=32 in., 18 in.}. Thus the provision requiring an embedment depth of ¾ of the cap depth would govern, requiring an embedment of 32 in. Cast-in-place bents currently require column bars to extend the cap depth less 6 to 9 in. This amounts to approximately 0.7 to 0.9 times the cap depth for most cases. Strands in pile caps use a smaller embedment depth. Minimum embedment depths should also account for vertical tolerances.

Minimum embedment depth provisions are likely not to govern for trestle pile caps, which tend to be more shallow.

5. *Cover*: For corrosion protection, it is recommended that top and side cover for connectors be at least 3 in. in both the cap and column or pile. Cover for duct protection should be considered satisfied when ducts are placed inside other cap and column or pile reinforcement. However, anchorage requirements still should be checked for connectors and ducts.
6. *Connector spacing*: See Section 6.3.7.2.

6.3.9.3.8 Guidelines for Application of Embedment Depth Requirements for Headed Bars in Grout Pockets

Guidelines listed in the previous section are also recommended in the application of the modified CCD method using Equation 6-8 to determine the required embedment depth for headed bars in grout pockets. In addition, the following provisions are recommended:

1. Concrete breakout capacity should be checked using groups of connectors that are expected to act in tension together. This can be determined from the neutral axis associated with the design axial load-moment conditions.
2. Concrete breakout may be checked in the transverse and longitudinal directions separately, although large biaxial moments may warrant a check of select connectors for combined eccentricities.
3. All connectors should be embedded to the depth required to ensure bar yield for the governing breakout surface.
4. The embedment depth need not exceed the development length requirement of Equation 6-6 for straight bar anchorage in grout pockets. This provision may be used for preliminary and/or final design.
5. Embedment depth for bars spaced closer than $2d_b$ need not be increased by 50 percent.

Further explanation of these provisions is given in Section 6.4.4.3.4.

6.3.10 Selection of Confining Reinforcement and Auxiliary Reinforcement

6.3.10.1 Introduction

The final step in the design process is the selection of confining reinforcement and auxiliary reinforcement for the connection region. Confining reinforcement refers to spiral reinforcement or closed ties placed around grout pockets or vertical ducts. Auxiliary reinforcement refers to the use of anchorage hardware or additional reinforcement in the cap top region for bolted connections.

6.3.10.2 Confining Reinforcement

6.3.10.2.1 Test Results

Confining reinforcement provided varying degrees of effectiveness during Phase 1-3 testing. Phase 1 pullout tests using single-line grout pockets demonstrated that, at an embedment depth of $8d_b$, spirals and welded wire fabric used as confining reinforcement increased strength and ductility and limited service-level cracking. Instead of the 2.5-in. spacing of #3 spirals used in Phase 1, Phase 2 used #3 spirals at an increased spacing of 4 in. The height of spiral reinforcement was limited to the 15-in. embedment depth for grout pocket connections, but was extended over the entire height of ducts for CVD and CBC specimens. In contrast to response observed in Phase 1, splitting cracks were not observed at any surface of the cap for Phase 2 or 3 tests, even for bar yield in failure tests. Phase 3 used the same spiral reinforcement as Phase 2, and again there was no evidence of splitting cracks in the connection region or connector slip, indicating that confining reinforcement contributed little, if at all, to connection ductility or strength. Based on the absence of splitting cracks in the connection region for Phase 2 and 3 tests and the fact that larger embedment depths will be used in design even if bars as large as #11's are used, the use of confining reinforcement is considered a conservative provision. (See Section 6.3.7.2 for further discussion.)

Nevertheless, designers are still encouraged to use confining reinforcement in the connection region because of the paucity of test data and because it has the potential to prevent deterioration of the joint by enabling a compression diagonal to form within the joint in cases of large moment transfer and to limit growth of potential splitting or inclined cracks in the connection region. Phase 2 and 3 construction demonstrated that placement of spiral reinforcement in the joint region was very simple and economical.

6.3.10.2.2 ACI and AASHTO Provisions

For planar frames like a bent, ACI-ASCE 352 requires that joint confinement be provided by a combination of column bars and ties in the joint region [6.16]. At least two layers of transverse reinforcement should be provided between the top and bottom levels of beam longitudinal reinforcement. The vertical center-to-center spacing of the transverse reinforcement should not exceed 12 in. for frames resisting gravity loads and 6 in. for frames resisting non-seismic lateral loads. Ties should satisfy Section 7.9 of ACI 318-99. It is recommended that longitudinal column reinforcement be uniformly distributed around the perimeter of the column core to improve confinement.

To ensure that the flexural capacity of the members can be developed without deterioration of the joint under repeated loadings, Section 7.9 of ACI 318-99 requires enclosure of reinforcement that terminates in connections. Enclosure must consist of external concrete or internal closed ties, spirals, or stirrups. To prevent deterioration due to shear cracking, Section 11.11.2 also requires that connections not confined by surrounding beams on all sides be confined by minimum lateral reinforcement, A_v, within the connection region for a depth equal to the deepest element framing into the connection. Minimum lateral reinforcement, A_v, is determined as follows [6.11, Section 11.5.5.3]:

$$A_v = \frac{50 b_w s}{f_y}$$

where: A_v = area of shear reinforcement, in^2

b_w = web width, in.

s = spacing of shear reinforcement, in.

f_y = specified yield strength of reinforcement, psi

AASHTO provisions for seismic design of beam-column joints include a requirement for column transverse reinforcement to continue a distance equal to one-half the maximum column dimension, but not less than 15 in., from the face of the column connection into the adjoining member. This is intended to prevent a plane of weakness at the cap to column interface [6.3, Section 7.6.4].

6.3.10.2.3 Recommendations

Based on the foregoing provisions, it is recommended that confining reinforcement be used in the joint region, as follows:

1. Minimum transverse confining reinforcement, A_v, should be no less than:

$$A_v = \frac{50 b_w s}{f_y} \qquad (6\text{-}9)$$

where: A_v = area of shear reinforcement, in^2

b_w = web width, in.

s = spacing of shear reinforcement, in.

f_y = specified yield strength of reinforcement, psi

2. Spirals or closed ties should be used. Spiral reinforcement, with a flat turn at the top and bottom, is recommended to facilitate construction.
3. Confining reinforcement should enclose all grout pockets or vertical ducts.
4. Confining reinforcement should extend the full height between the top and bottom longitudinal reinforcement in the cap, regardless of the embedment depth. This is conservative and facilitates construction.
5. The vertical center-to-center spacing of the transverse reinforcement should not be less than 2 in. nor greater than 6 in. The lower bound helps ensure adequate flow of concrete between reinforcement; the upper bound is intended to help ensure effectiveness of the reinforcement.

For a 33-in. wide cap, the minimum reinforcement provision requires a 3/8-in. spiral at a 4-in. pitch or #3 bars at 4 in. on center.

6.3.10.3 Auxiliary Reinforcement

Designers should also detail auxiliary reinforcement, such as anchorage hardware and reinforcement in the cap top region for bolted connections. Although non-proprietary anchorage hardware such as bearing plates and nuts may suffice for bolted connections, designers can also use proprietary anchorage systems such as those developed by Dywidag or Williams for post-tensioning.

Phase 2-3 tests indicated minimal bearing forces at cap top anchorages, due to the development of bond along the bolts. Designers may choose to provide bursting and spalling reinforcement in the cap top region of bolted connections, particularly if post-tensioning is used.

6.4 DEVELOPMENT OF DESIGN EQUATIONS FOR CONNECTOR ANCHORAGE

6.4.1 Introduction

In this section, design equations for connector anchorage are developed for the four connection cases investigated in testing: 1) straight bars in grout pockets, 2) straight bars in grouted vertical ducts, 3) headed bars in grouted vertical ducts, and 4) headed bars in grout pockets. Because yielding of steel is a more predictable and ductile failure mode than concrete failure, embedment design is intended to ensure connector yield is the governing failure mode.

Some facets of connection behavior for these four cases exhibited similarities to cast-in-place anchors, while other aspects were significantly different. Important differences in behavior were also evident in the various phases of testing. For example, bent cap-to-column connection tests conducted in Phases 2 and 3 demonstrated that the transfer of forces through a connection produces a less severe condition for anchorage than observed in Phase 1 pullout tests. Thus, the approach used to develop design equations for embedment depth initially follows traditional approaches for cast-in-place anchors: a uniform bond stress model for straight bars and a concrete breakout surface model for headed bars based on the Concrete Capacity Design (CCD) method. The uniform bond stress model provides a simple development length requirement for design, whereas the concrete breakout surface approach provides a simple method to determine connector capacity for an assumed breakout surface. Test data is used to calibrate such expressions, and, where necessary, to modify the design approach.

Prior to testing, straight bar development lengths were thought to be potentially excessive for a precast bent cap system. Thus, 20 of the first 24 Phase 1 tests were conducted using headed bars. After straight bar anchorage was found to be sufficient, subsequent tests focused on straight bars. Nevertheless, a limited number of straight bar tests are available for development of design equations for straight bar anchorage. To account for this paucity of test data, considerable conservatism is incorporated in the development of all design equations. Portions of the subsequent development build upon an initial review of test data provided in Reference 6.19.

6.4.2 Straight Bar Tests

6.4.2.1 Previous Pullout Tests

As mentioned in Section 1.5.3, two recent pullout test programs have been conducted on grouted bars, one using straight reinforcing bars and the other using straight and headed threaded rods [6.20, 6.21]. Reference 6.20 developed a design equation for the bond strength of grouted bars that is proportional to the embedment depth and the square root of the concrete compressive strength. Splitting was the most common failure mode and was usually accompanied by failure at the bar-grout interface. Some pullout failures developed at the grout-concrete interface. Reference 6.21 developed an expression for the mean tensile strength at bond failure proportional to the embedment depth and the mean bond stress at the grout/anchor interface. These tests were conducted using Masterflow 928 and Masterflow 885.

The ACI 318-99 tension development length equation [6.11, Equation 12-1] accounts for splitting and pullout failures of standard reinforcing bars embedded in cast-in-place concrete. For the large cover used in grout pocket and grouted vertical duct specimens, a pullout failure, accompanied by splitting, is expected. ACI 318-89 included an explicit check for pullout failure formulated as an additional minimum development length requirement. This expression, based on pullout tests of bars that developed a average bond strength of $9.5\sqrt{f'_c}/d_b$, requires a development length as follows:

$$l_d = \frac{0.03 d_b f_y}{\sqrt{f'_c}} \qquad (6\text{-}10)$$

where: l_d = development length, in.

d_b = nominal diameter of bar, in.

f_y = specified yield strength of connector, psi

f'_c = specified concrete compressive strength, psi

As shown in Chapter 1, the current ACI 318-99 equation reduces to this same equation for large cover.

6.4.2.2 Brief Review of Test Results

The four Phase 1 tests conducted on straight bars embedded in grout pockets exhibited bar pullout or bar yield, accompanied by significant splitting cracks (Sections 3.3.4.4 and 3.4.3.3). Bar yield was achieved at an embedment depth of only 12d_b due to confinement effects of the surrounding concrete (including cap reinforcement). Use of spiral and welded wire confinement around pockets increased ductility. Five of the vertical duct tests using straight bars exhibited a pullout failure of the grout-bar mass from the duct, associated with significant splitting. Confinement effects due to the surrounding concrete enhanced anchorage. Bar yield was achieved at an embedment depth of 13d_b. The average concrete compressive strength, f'c, was approximately 5.5 ksi and the average modified grout cube strength was 6.6 ksi.

Phase 2 and 3 grout pocket and grouted vertical duct tests used straight, epoxy-coated #9 bars at an embedment depth of 13d_b. A realistic cover and distance between bars was used for 2/2 connector arrangements. The average f'c was approximately 6 ksi; grout strength varied from approximately 4 to 6 ksi. Connection distress under proof loads was minor, but failure tests demonstrated that the anchorage was sufficient to develop 1.2 times the bar yield strength. Splitting cracks associated with pullout in Phase 1 tests did not appear in the connection region, suggesting little contribution of the spiral confining reinforcement to ductility or strength. The formation of vertical splitting cracks on the tension face of columns indicated the need for adequate anchorage of connectors into columns or piles.

6.4.2.3 Required Development Length for Grout Pockets

A uniform bond stress model is used to establish the required development length for straight bars. Based on test data, a conservative value for the ultimate bond strength of connectors embedded in grout pockets is determined. This value is then used to establish the minimum development length requirement to achieve 1.25 times the specified yield strength of the bar. Design development lengths also incorporate a strength reduction factor.

Table 6.4 lists the key variables and results of the four Phase 1 straight bar tests for grout pockets. Based on the maximum applied load per bar, the average bond stress, u, is determined as the load divided by the nominal surface area:

$$u = \frac{P}{\pi d_b h_{ef}} \qquad (6\text{-}11)$$

where: u = average bond stress, psi

P = applied load, lbs.

d_b = nominal diameter of bar, in.

h_{ef} = embedment depth, in.

Table 6.4 Average Bond Strength—Phase 1 Straight-Bar Grout Pocket Tests

Test ID	Bars no.-size	Bars f_y (ksi)	A_b in^2	d_b in	h_{ef} in	f'_c (ksi) concr	f'_c (ksi) grout	P_{yield} kips	P_{max} kips	u_{max}^{A} psi	$u_{max}/\sqrt{f'_c}$	$u_{max}/\sqrt{f'_{cg}}$	$u_{dgn}/\sqrt{f'_c}$^{C}
SL13	1-#8	69	0.79	1.00	12	5.2	6.9	52	56	1485	20.6	17.9	19.1
SL14	1-#8	69	0.79	1.00	18	5.1	6.9	52	73	1291	18.1	15.5	15.2
DL05^B	2-#6	65	0.44	0.75	9	5.7	6.3	-	24	1132	15.0	14.3	13.7
DL06	2-#6	65	0.44	0.75	9	5.8	6.4	27	28	1320	17.3	16.5	14.9
									ave	**1307**	**17.8**	**16.0**	**15.7**
									std dev	**145**	**2.3**	**1.5**	**2.3**

Footnotes

A. $u=P/(\pi d_b h_{ef})$

B. Bar yield was not achieved

C. Based on bond stress at beginning of significant head slip

Phase 1 straight bar tests exhibited cracking in both the grout pocket and surrounding concrete, indicating that, to some degree, the tensile strength of both the grout and concrete affect behavior. However, pullout failure was always preceded by significant splitting of the surrounding concrete, which acted to confine the grout pocket. Thus, the tensile strength of the concrete, rather than that of the grout or some combination of both, is used in development of a design equation. This assumes that the grout will possess sufficient strength to prevent a failure due to splitting and pullout in the pocket alone. Sufficient grout strength is expected because the grout specification described in Chapter 7 requires a modified grout cube strength 1000 psi greater than the concrete compressive strength. For these four tests, the modified grout cube strength exceeded the concrete compressive strength by an average of 1100 psi on test day.

Following the common assumption that the tensile strength of concrete is proportional to the square root of its compressive strength, the normalized bond stress is defined as the average bond stress divided by the square root of the concrete compressive strength, $u/\sqrt{f'_c}$. Table 6.4 shows the average normalized bond stress at maximum load, $u_{max}/\sqrt{f'_c}$, to be 17.8. For comparison, Table 6.4 shows $u_{max}/\sqrt{f'_{cg}}$, the normalized bond stress based on grout cube strength, to be 10 percent less. In addition, Table 6.4 lists the average normalized bond stress for design, $u_{dgn}/\sqrt{f'_c}$.

Design values were conservatively based on the value of $u/\sqrt{f'_c}$ at the beginning of significant end slip. Figure 6.13 shows a plot of $u/\sqrt{f'_c}$ vs. end slip. SL13 used an embedment of $12d_b$ and was loaded twice, the second time to produce pullout failure. The end slip record for SL13 shows significant slip during the final two load increments, corresponding to a normalized bond stress of 19.1 prior to significant end slip. The lead slip measurement shown in Figure 6.13 confirmed bar slip at this load. The use of a larger embedment of $18d_b$ for SL14 produced a larger capacity prior to failure. The lead slip is plotted prior to bar yield because the head slip record was unreliable. Significant splitting cracks and presumably the beginning of significant end slip were observed at $u/\sqrt{f'_c}$ equal to 15.2. DL05 and DL06 exhibited sudden slip similar to SL13. For DL06, spiral confinement enhanced ductility and caused the entire confined pocket to act to some extent as a whole, extending cracks further into the specimen than other straight bar tests. Figure 6.13 shows normalized design bond stresses for DL05 and DL06 of 13.7 and 14.9, respectively.

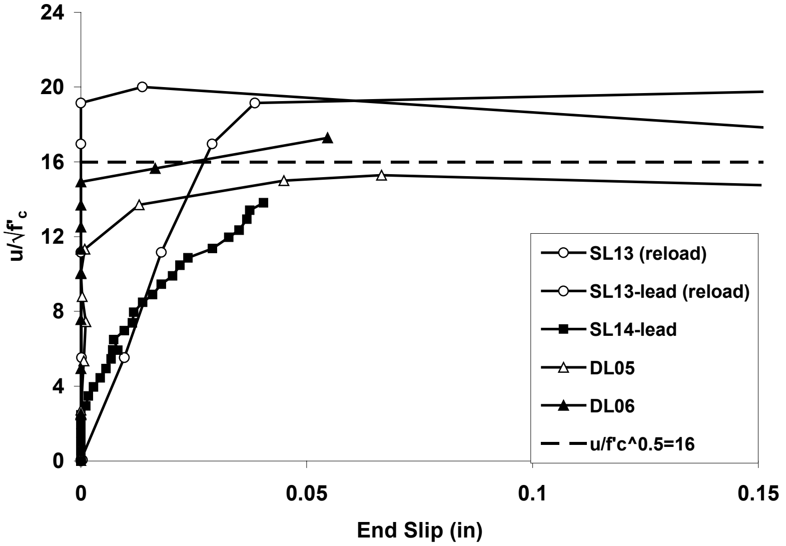


Figure 6.13 Normalized Bond Stress, $u/\sqrt{f'_c}$ vs. End Slip—Grout Pocket Straight-Bar Tests

Table 6.4 shows the normalized average bond stress for design, $u_{dgn}/\sqrt{f'_c}$, to be 15.7. The dashed horizontal line, shown in Figure 6.13 for $u_{dgn}/\sqrt{f'_c}$ shows that this value provides a reasonably conservative bound on the test data, ignoring additional strength achieved during the highly non-linear stage of bar slip up to failure. The average bond stress at maximum load is 13 percent larger than the design value.

A value of 15.7 for $u_{dgn}/\sqrt{f'_c}$ corresponds to an assumed maximum bond strength of $15.7\sqrt{f'_c}$ for design. Based on Equation 6-11, this normalized bond stress can be used to determine a required embedment depth or development length:

$$u = 15.7\sqrt{f'_c} = \frac{P}{\pi d_b h_{ef}}$$

To account for the possibility of overstrength and strain hardening, a 25 percent increase in the specified yield strength is assumed. The maximum bar force at the design bond stress is:

$$P = (1.25f_y)\frac{\pi d_b^2}{4} = \frac{f_y \pi d_b^2}{3.20}$$

Therefore

$$u = 15.7\sqrt{f'_c} = \frac{f_y \pi d_b^2}{3.20 d_b h_{ef}}$$

Reducing terms results in the following:

$$15.7\sqrt{f'_c} = \frac{d_b f_y}{3.20 h_{ef}}$$

Rearranging terms and substituting l_d for h_{ef} gives

$$l_d = \frac{d_b f_y}{3.20 \times 15.7\sqrt{f'_c}} \text{ or}$$

$$l_d = \frac{d_b f_y}{50.2\sqrt{f'_c}} \qquad (6\text{-}12)$$

For use in design, the required development length should be divided by the strength reduction factor, ϕ. A strength reduction factor of 0.9 is recommended for use to reflect the importance of connection reliability as well as the paucity of data on which Equation 6-12 is based. However, use of a strength reduction factor smaller than 0.9 is not deemed necessary for the following reasons:

1. Compounded safety factors in Equation 6-12 are included, by using a 25 percent increase in f_y and discounting additional strength beyond initial stages of bar slip.
2. Phase 2 and 3 straight bars achieved approximately 1.2 times the yield stress in grout pocket connections at an embedment depth of $13d_b$.
3. Concrete compressive strengths for actual caps will typically exceed the specified concrete strength assumed in Equation 6-12.

4. A ductile failure mode governed by connector yield is expected.

Multiplying Equation 6-12 by 1/0.9, or 1.11, and rounding off the coefficient leads to the following design requirement for development length of straight bars in grout pockets:

$$l_d = \frac{0.022 d_b f_y}{\sqrt{f'_c}} \qquad (6\text{-}13)$$

where: l_d= development length, in.

d_b=nominal diameter of bar, in.

f_y=specified yield strength of connector, psi

f'_c=specified concrete compressive strength of the bent cap, psi

This is the same as Equation 6-6 in Section 6.3.9.3.1. Since all tests were conducted on epoxy-coated bars, this equation may be conservatively applied to both coated and uncoated connectors. For the common case of a Grade 60 reinforcing bar with f'_c of 3600 psi, the required development length would be $22d_b$, or approximately 1.7 times that used to develop connectors in Phases 1 and 2.

Table 6.5 uses Equation 6-13 to determine the required development length and compares the predicted anchorage resistance of the bar to test results. Based on the assumption of uniform bond stress along the entire bar length, the maximum anchorage force predicted by Equation 6-13, P_{pred}, would be the bar force at yield times the ratio of the actual embedment depth to the required development length, or

$$P_{pred} = \frac{h_{ef}}{l_d} A_s f_y$$

The ratio of P_{test}/P_{pred} represents the safety factor built into the design equation. Table 6.5 shows the average safety factor using Equation 6-13 to be 1.6.

Table 6.5 Comparison of Predicted and Actual Capacity—Phase 1 Straight-Bar Grout Pocket Tests

Test ID	A_b in^2	d_b in	f_y ksi	f'_c ksi	h_{ef} in	l_d^{A} in	P_{pred}^{B} kips	P_{test} kips	P_{test}/P_{pred}
SL13^C	0.79	1.00	69	5.2	12	21.1	31	56	1.80
SL14^C	0.79	1.00	69	5.1	18	21.3	46	73	1.58
DL05	0.44	0.75	65	5.7	9	14.2	18	24	1.32
DL06	0.44	0.75	65	5.8	9	14.1	18	28	1.53

Footnotes **ave** **1.6**

A. $l_d=0.022f_yd_b/\sqrt{f'_c}$

B. $P_{pred}=h_{ef}/l_d(A_sf_y)$, which represents the maximum tensile capacity predicted for the given embedment depth

C. Test terminated before maximum load was achieved

The ACI 318-89 development length provision shown in Equation 6-10 requires a development length approximately 1/3 more than that required by Equation 6-13. Although the assumed bond strength for connectors in grout pockets was approximately 2/3 greater than that assumed for cast-in-place reinforcing bars, the greater conservatism incorporated in Equation 6-13 resulted in only a 33 percent difference. The general equation for development length, Equation 12-1, found in Section 12.2.3 of ACI 318-99 includes a limit to safeguard against pullout failure, which is the same provision as Equation 6-10. However, by including a coating factor of 1.2 for epoxy-coated bars, Equation 12-1 requires a development length approximately 2/3 larger than Equation 6-13.

6.4.2.4 Required Development Length for Grouted Vertical Ducts

Table 6.6 lists the key variables and results of the six Phase 1 straight bar tests for grouted vertical ducts. As in the previous section, the average bond stress and normalized bond stress are tabulated. Phase 1 straight bar tests in grouted vertical ducts exhibited less cracking in the grout than did grout pocket tests, as well as less pronounced effects of splitting on bar force distribution. Because adequate bond developed at the interfaces between the bar, grout, duct, and concrete, the bar and grout mass in the vertical duct tended to act together as a single unit in producing splitting cracks in the surrounding concrete. Such response resulted even though the modified grout cube strength was an average of 1200 psi less than the concrete compressive strength. One exception was VD04 for which Euclid Hi-Flow grout with a very low compressive strength was used, resulting in pullout associated with significant splitting of the grout in the duct as well as in the surrounding concrete (Section 3.5.4.2).

Based on this general behavior, the tensile strength of the concrete is considered to be the more significant parameter affecting pullout strength. This is expected to be the case in practice as well because the grout specification requires that the modified grout cube strength exceed the specified concrete strength by 1000 psi. Concrete splitting is accounted for in Table 6.6 by use of the square root of the concrete compressive strength in the normalized bond stress, $u/\sqrt{f'_c}$. Table 6.6 shows the average value of $u/\sqrt{f'_c}$ to be 13 percent less than $u/\sqrt{f'_{cg}}$.

Figures 6.14 and 6.15 show plots of $u/\sqrt{f'_c}$ vs. duct strain and $u/\sqrt{f'_c}$ vs. end slip, respectively. As mentioned in Section 3.5, large duct strains corresponded to dilation associated with large slip of the grout-bar mass within the duct leading to pullout failure. Duct strains corresponded to the maximum strain record for the duct, usually at the 12-in. location in the spiral direction. Two of the six tests, VD01 and VD04, exhibited a clear pullout failure. Figures 6.14 and 6.15, as well as the figures in Section 3.5, show that three of the remaining four tests exhibited end slip, duct strains, and splitting cracks characteristic of pullout failure, despite the fact that loading was discontinued prior to complete pullout failure. VD07, however, was discontinued at a load preceding significant splitting and end slip. Thus, Table 6.6 shows a conservative normalized bond stress for VD07.

Table 6.6 lists the normalized bond stress at maximum loads and at conservative design loads based on Figures 6.14 and 6.15. Values of $u_{dgn}/\sqrt{f'_c}$ correspond to the beginning of significant end slip, as determined from the figures. Table 6.6 shows the average of $u_{dgn}/\sqrt{f'_c}$ to be 14.1 for the six tests, or 14.3 without VD07. The dashed horizontal line shown in the figures shows that use of $u_{dgn}/\sqrt{f'_c}$ equal to approximately 14 provides a reasonably conservative bound on the test data because it ignores additional strength achieved during the highly non-linear stage of bar slip up to failure. Although the design strength for VD04 is one standard deviation below the average due to excessively low grout strength in the duct, it is conservatively included in the average design strength. The average normalized bond stress at maximum load was 17.9, which is 25 percent greater than the design value of 14.3.

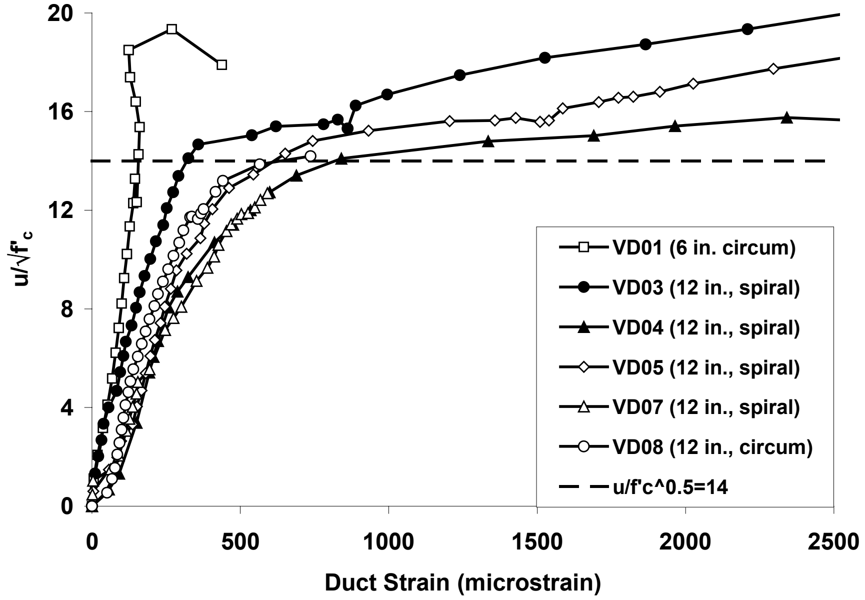


Figure 6.14 Normalized Bond Stress, u/√f'$_c$ vs. Duct Strain— Grouted Vertical Duct Straight-Bar Tests

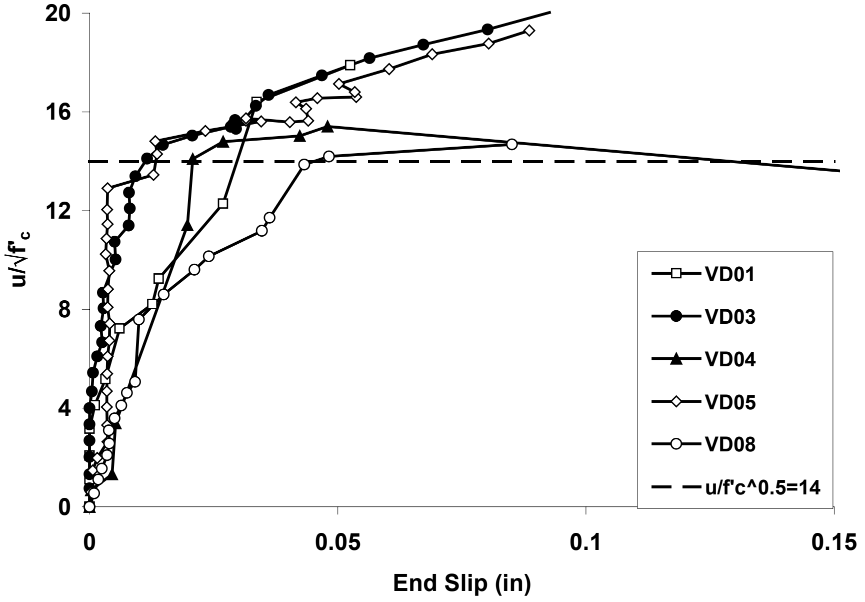


Figure 6.15 Normalized Bond Stress, u/√f'$_c$ vs. End Slip— Grouted Vertical Duct Straight-Bar Tests

Table 6.6 Average Bond Strength—Phase 1 Straight Bar Vertical Duct Tests

Test ID[C]	Bars no.-size	Bars f_y (ksi)	A_b in^2	d_b in	h_{ef} in	f'_c (ksi) concr	f'_c (ksi) grout	P_{yield} kips/bar	P_{max} kips/bar	u_{max}[A] psi	$u_{max}/\sqrt{f'_c}$	$u_{max}/\sqrt{f'_{cg}}$	$u_{dgn}/\sqrt{f'_c}$[B]
VD01[D]	1-#11	59	1.56	1.41	12	5.4	4.2	-	76	1430	19.5	22.1	16.4
VD03	1-#11	59	1.56	1.41	18	5.6	5.7	92	119	1492	19.9	19.8	14.6
VD04	1-#11	59	1.56	1.41	18	5.6	3.1	94	94	1179	15.8	21.2	12.7
VD05	1-#11	59	1.56	1.41	18	5.5	3.8	93	114	1430	19.3	23.2	14.8
VD07[E]	1-#11	59	1.56	1.41	24	5.5	5.2	94	100	941	12.7	13.0	12.6
VD08	1-#11	59	1.56	1.41	24	5.5	4.5	93	118	1110	15.0	16.5	13.2
								w/o VD07:	**ave**	**1328**	**17.9**	**20.5**	**14.3**
								w/oVD07:	**std dev**	**171**	**2.3**	**2.6**	**1.5**
								w/VD07:	**ave**	**1264**	**17.0**	**19.3**	**14.1**

Footnotes

A. $u=P/(\pi d_b h_{ef})$

B. Based on bond strength at beginning of significant head slip

C. All tests terminated before maximum load achieved

D. Bar yield was not achieved

E. Maximum load significantly less than capacity

Following the same development used for straight bars in grout pockets, and assuming again a 25 percent increase in the specified yield strength of bars, the following equations derive the development length for straight bars in grouted ducts:

$$u = 14.3\sqrt{f'_c} = \frac{P}{\pi d_b h_{ef}}$$

P is obtained from

$$P = (1.25f_y)\frac{\pi d_b^{\,2}}{4} = \frac{f_y \pi d_b^{\,2}}{3.20}$$

Thus

$$u = 14.3\sqrt{f'_c} = \frac{f_y \pi d_b^{\,2}}{3.20 d_b h_{ef}}$$

Reducing terms results in the following

$$4.3\sqrt{f'_c} = \frac{d_b f_y}{3.20 h_{ef}}$$

Rearranging terms and substituting l_d for h_{ef} gives

$${}_d = \frac{d_b f_y}{3.20 \times 14.3\sqrt{f'_c}} \text{ or}$$

$$l_d = \frac{d_b f_y}{45.8\sqrt{f'_c}} \qquad (6\text{-}14)$$

Based on the same reasoning used for straight bar anchorage in grout pockets, a strength reduction factor of 0.9 is recommended. Multiplying Equation 6-14 by 1/0.9, or 1.11, and rounding off the coefficient leads to the following design requirement for development length of a straight bar in a grouted vertical duct:

$$l_d = \frac{0.024 d_b f_y}{\sqrt{f'_c}} \qquad (6\text{-}15)$$

This is the same as Equation 6-7 in Section 6.3.9.3.2. Since all tests were conducted on epoxy-coated bars, this equation may be conservatively applied to both coated and uncoated connectors. For the common case of a Grade 60 reinforcing bar with f'_c of 3600 psi, the required development length is $24d_b$, or approximately 1.8 times that used to develop connectors in Phases 1 and 2.

Table 6.7 uses Equation 6-15 to determine the required development length and compares the predicted anchorage resistance of the bar to test results. Table 6.7 shows the average safety factor, P_{test}/P_{pred}, to be 1.7 (excluding VD07).

Table 6.7 Comparison of Predicted and Actual Capacity—Phase 1 Straight-Bar Vertical Duct Tests

Test ID[C]	**A_b in^2**	**d_b in**	**f_y ksi**	**f'_c ksi**	**h_{ef} in**	**l_d**[A] **in**	**P_{pred}**[B] **kips**	**P_{test} kips**	**$P_{test/}P_{pred}$**
VD01[D]	1.56	1.41	59	5.4	12	27.2	41	76	1.87
VD03	1.56	1.41	59	5.6	18	26.7	62	119	1.92
VD04	1.56	1.41	59	5.6	18	26.7	62	94	1.51
VD05	1.56	1.41	59	5.5	18	26.9	62	114	1.85
VD07	1.56	1.41	59	5.5	24	26.9	82	100	-[E]
VD08	1.56	1.41	59	5.5	24	26.9	82	118	1.44

Footnotes **ave** **1.7**

A. $l_d=0.024f_yd_b/\sqrt{f'_c}$

B. Ppred=$h_{ef}/l_d(A_sf_y)$, which represents the maximum tensile capacity predicted for the given embedment depth

C. All tests terminated before maximum load was achieved

D. Bar pullout before bar yield

E. Maximum load significantly less than capacity

Equation 6-15 is larger than Equation 6-13 by a factor of 24/22, or 1.09. The reason for the difference is the slightly larger design bond stress used for grout pockets, which was affected by a certain amount of "subjectiveness" in processing the data. Given the difference in behavior for grout pockets vs. grouted vertical ducts observed in tests as well as potential scatter in data, the difference is considered to be very small. The small difference also reflects the significant influence of concrete splitting on pullout failure for both connection types. For epoxy-coated bars, Section 12.2.3 of ACI 318-99 requires a development length 50 percent longer than Equation 6-15. This difference reduces to 25 percent for uncoated bars.

6.4.3 Headed Bars in Grouted Vertical Ducts

Phase 1 tests demonstrated that the use of a duct in a precast connection significantly affects the failure mode when headed bars are used. Thus, fundamentally different approaches are used for headed bars when used in grout pockets vs. grouted vertical ducts.

Two sets of tests were conducted to compare pullout behavior for straight and upset-headed bars grouted in vertical ducts (Section 3.5.4.3). VD01 and VD02 used an embedment depth of 12 in., and VD03 and VD06 used a depth of 18 in. In contrast to the straight bar used for VD01, the VD02 headed bar achieved a capacity 20 percent larger (Figure 3.88) and with much greater ductility, although reduced stiffness developed due to effects of splitting cracks. Figure 3.89 shows the development of large duct strains near the head, indicating large local stresses around the head. Figure 3.90 showed that the head carried 2/3 of the maximum load and therefore provided significant anchorage. At the deeper 18-in. embedment, the straight and headed bars achieved yield and portrayed very similar response (Figures 6.14 and 3.91). Thus, when connectors are embedded sufficiently to develop yield, the head tends to carry only a small portion of load, resulting in little difference in response.

Although headed bars may provide the additional anchorage capacity to enable shallow embedded bars to achieve yield, such a benefit cannot be accurately quantified without additional research. Because connectors in a precast bent cap are designed to ensure connectors achieve yield, it is not recommended that a designer attempt to justify a reduction in development length for grouted vertical duct connections by use of a headed bar. Equation 6-15, therefore, should be used for both straight and headed bars grouted in vertical ducts. For cases in which a straight bar is embedded adequately according to Equation 6-15, a designer may choose to use a headed bar to provide reserve connection strength.

6.4.4 Headed Bars in Grout Pockets

6.4.4.1 Previous Tests

As discussed in Chapter 1, the Concrete Capacity Design (CCD) Method [6.22,6.23] provides a convenient and accurate approach to determine the tensile capacity of cast-in-place concrete anchors. Reference 6.24 verified the accuracy of this approach for determining the concrete breakout capacity in pullout tests of T-headed headed bars. Reference 6.21 used the CCD Method calibrating a design equation for threaded rod anchors with hex-nut head that were grouted in small diameter holes with Masterflow 928.

Using beam specimens, Reference 6.25 developed an equation for the development length of headed bars grouted in small diameter holes. For Grade 60 bars and two bar diameters of cover, the design equation simplified to:

$$l_d = \frac{700 d_b}{\sqrt{f'_c}} \qquad (6\text{-}16)$$

The coefficient of 700 is approximately 60 percent of that required by ACI for hooked bars. Equation 6-16 requires a development length approximately 50 percent of that required by Equation 6-13 for straight bars in grout pockets and by Equation 6-15 for straight or headed bars in grouted vertical ducts. Equations 6-13 and 6-15 compare closely to ACI 318-99 development length requirements for hooked bars.

6.4.4.2 Brief Summary of Test Results

All epoxy-coated bars used in Phase 1 single-line and double-line grout pocket tests failed by concrete breakout or bar yield. Of the twenty-four tests, fourteen achieved a concrete breakout failure with some preceded by bar yield. Two other tests, SL07 and SL08, exhibited head slip and splitting associated with the beginning stages of failure (Figure 3.47) and are also conservatively included. These sixteen tests, listed in Table 6.8, are the basis for the development of a design equation for embedment of headed bars.

Yield was achieved in single bar tests for embedment depths of $12d_b$ for #8 bars and $8d_b$ for #6 bars. Confinement effects due to the concrete and longitudinal bars around the grout pocket enhanced anchorage. Excellent interlock between the grout pocket and concrete interface was achieved without surface roughening. Multiple bars achieved approximately the same capacity as the corresponding single bar specimen. Compared to the cast-in-place bars of SL16, grouted bars in SL15 achieved a capacity 20 percent less and exhibited softer response and more extensive splitting cracks. Confining reinforcement increased capacity 50 percent over the unconfined specimen and increased ductility.

Due to the adequate anchorage of straight bars in Phase 1 tests, only straight bars were used in Phases 2 and 3.

6.4.4.3 Connector Anchorage for Grout Pockets

This section develops a design expression for anchorage of headed bars in grout pockets using the CCD Method. Modifications to the basic CCD equation for concrete breakout are introduced to account for differences in behavior and failure modes for grouted connectors. For example, failure surfaces were significantly affected by the development of cracks, particularly at pocket corners. Provisions are included to enable the designer to use the modified CCD design strength equation to solve for the required development length to achieve 1.25 times the specified yield strength of the bar.

Table 6.8 Comparison of Predicted and Actual Capacity—Phase 1 Headed Bar Grout Pocket Tests

Test	Bars	h_{ef}	f'_c (ksi)		P_{max}	A_N	A_{NO}	A_N/A_{NO}	c_{min}	$1.5h_{ef}$	Ψ_E	Ψ_C	P_{CCD}[A]	P_{max}/P_{CCD}	f'_{c1}/f'_c[C]	P_{max}/P_{CCD}
ID	no.-size	in	concr	grout	kips/bar	in^2	in^2		in	in			kips/bar	$(\Psi_C=1.0)$		$(\Psi_C=0.78)$
SL01	1-#8	6	5.4	6.9	36	324	324	1.00	12	9	1.00	1	43	0.83	1.07	1.07
SL02	1-#8	6	5.4	6.4	37	324	324	1.00	12	9	1.00	1	43	0.86	1.04	1.10
SL03	1-#8	12	5.2	6.5	60	864	1296	0.67	12	18	0.90	1	73	0.83	1.02	1.06
SL04	1-#8	12	5.4	6.3	63	864	1296	0.67	12	18	0.90	1	74	0.85	1.01	1.09
SL05	1-#8	9	6.3	7.0	46	648	729	0.89	12	13.5	0.97	1	74	0.62	1.01	0.80
SL06	1-#8	9	6.5	7.1	45	648	729	0.89	12	13.5	0.97	1	75	0.60	1.01	0.77
SL07	1-#8	18	5.4	7.0	70	1296	2916	0.44	12	27	0.83	1	90	0.78	1.02	1.00
SL08	1-#8	18	5.5	8.5	64	1296	2916	0.44	12	27	0.83	1	91	0.71	1.03	0.91
SL09	1-#6	4	5.5	7.4	21	144	144	1.00	12	6	1.00	1	24	0.88	1.18	1.13
SL11	1-#6	6	5.2	6.9	34	324	324	1.00	12	9	1.00	1	42	0.80	1.08	1.03
SL12	1-#6	6	5.2	6.3	35	324	324	1.00	12	9	1.00	1	42	0.83	1.05	1.06
SL15	2-#8	12	5.0	5.3	32	1056	1296	0.81	12	18	0.90	1	43	0.74	1.00	0.94
DL01	2-#6	6	5.0	4.2	24	432	324	1.33	8	9	0.97	1	27	0.90	0.97	1.15
DL02	2-#6	6	5.1	5.9	20	442	324	1.36	8	9	0.97	1	28	0.72	1.03	0.93
DL03[B]	2-#6	6	5.6	5.8	22	442	324	1.36	8	9	0.97	1	29	0.76	1.01	0.97
DL04[B]	2-#6	6	5.6	5.4	24	442	324	1.36	8	9	0.97	1	29	0.83	0.99	1.06
													ave	**0.78**	**1.03**	**1.00**
													std dev	**0.09**	**0.05**	**0.11**
													max	**0.90**	**1.18**	**1.15**
													min	**0.60**	**0.97**	**0.77**
													r^2	**0.92**		**0.92**

Footnotes

A. $P_{CCD}=(A_n/A_{no}\Psi_E\Psi_C)40f'_c{}^{0.5}h_{ef}{}^{1.5}$ ($h_{ef}\leq 11$ in.)

$P_{CCD}=\max((An/Ano\Psi_E\Psi_C)40f'c^{0.5}hef^{1.5}, A_n/A_{no}\Psi_E\Psi_C)27f'_c{}^{0.5}h_{ef}{}^{1.5})$ ($h_{ef}>11$ in.)

B. Confining reinforcement ineffective

C. $f'_{c1}=[f'_c(1-A_g/A_n)]+f'_{cg}(A_g/A_n)$

6.4.4.3.1 General CCD Equation

The CCD equation for the nominal concrete breakout strength for a group of fasteners in tension, P_{cbg}, is shown below:

$$P_{cbg} = \frac{A_N}{A_{No}} \Psi_E \Psi_C P_b \qquad (6\text{-}17a)$$

where: P_{cbg} = nominal concrete breakout strength in tension of a group of fasteners, lbs

A_N = projected concrete failure area of a fastener or group of fasteners, in^2, not to exceed nA_{No}, where n is the number of tensioned fasteners in the group

A_{No} = projected concrete failure area of one fastener, when not limited by edge distance or spacing, in^2
= $9h_{ef}^2$

Ψ_E = modification factor to account for edge distances smaller than $1.5h_{ef}$

= 1 if $c_{min} \geq 1.5h_{ef}$

= $0.7 + 0.3\dfrac{c_{min}}{1.5h_{ef}}$ if $c_{min} < h_{ef}$

Ψ_C = modification factor to account for grout pocket connection cracking

P_b = basic concrete breakout strength in tension of a single fastener, lbs

= $40\sqrt{f'_c}\,h_{ef}^{1.5}$ $h_{ef} \leq 11$ in. (6-17b)

= $27\sqrt{f'_c}\,h_{ef}^{5/3}$ $h_{ef} > 11$ in. (6-17c)

h_{ef} = embedment depth, in.

c_{min} = smallest of the edge distances that are less than or equal to $1.5h_{ef}$, in.

The coefficients of 40 and 27 are used to calibrate Equations 6-17b and 6-17c, respectively, to the mean breakout strength. Smaller values of 24 and 16, respectively, are used for design. The term A_N/A_{No} is used to account for a breakout surface associated with a group of fasteners.

Pullout, based on crushing at the anchor head in CB-30, and side blowout should also be checked, but are unlikely for typical conditions, especially when the contribution of bearing on lugs of reinforcing bars is accounted for. The designer, however, should check these failure modes using Reference 6.23.

6.4.4.3.2 Comparison of Grout Pocket Test Results to CCD

Table 6.8 lists the key variables and results for the headed bar tests and compares the maximum applied load during the test, P_{max}, to the capacity predicted by the CCD Method, P_{CCD}, using the ratio P_{max}/P_{CCD}. The average value of P_{max}/P_{CCD} is shown to be 0.78, i.e., an under-prediction of 22 percent on average. Figure 6.16 plots P_{max} vs. P_{CCD}, showing that maximum load is less than that predicted using the CCD Method. More importantly, the linearity of data points in Figure 6.16 shows a strong dependence between P_{max} and P_{CCD}. In fact, as shown in Table 6.8, the coefficient of determination (Pearson r^2 term) is 0.92, with a corresponding correlation, r, of 0.96. In addition, a small standard of deviation resulted, confirming a fairly small amount of scatter in the data.

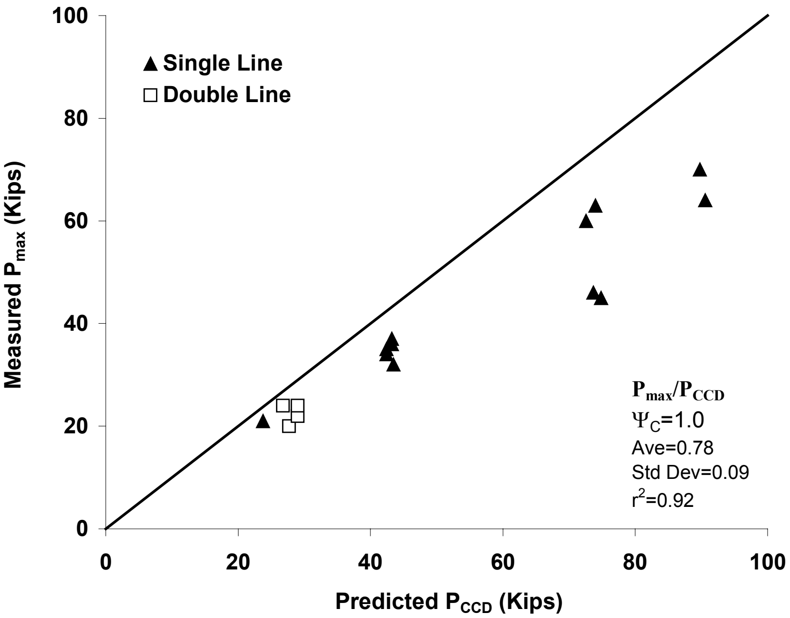


Figure 6.16 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests

The smaller capacity achieved in grout pocket tests compared to CCD predictions is expected because the CCD Method is based on anchorage capacity of cast-in-place fasteners. Phase 1 comparison tests, SL15 and SL16, exhibited a 20 percent larger capacity for the cast-in-place bars compared to bars embedded in grout pockets (see Figures 3.55 and 3.56). While pullout tests on cast-in-place bars produced the sudden formation of a breakout cone, grouted connectors exhibited a markedly softer load-slip behavior, reflecting the gradual development of cracks and loss of confinement from the surrounding concrete that preceded the breakout failure. The greater disturbance of the tensile stress fields, especially at the pocket corners, led to a decreased surface area available for transfer of tensile forces, and thus a smaller capacity.

The CCD method accounts for the influence of cracks on capacity by use of the Ψ_c factor, which is taken as 1.0 for fasteners embedded in concrete expected to be cracked in the fastener region at service loads. As shown in Table 3.6, cracks at loads of approximately 60 to 80 percent of P_{max} (or P_{yield}) were often 0.013 in. or larger. More importantly, during loading of the specimens to failure, disturbance of the tensile stress field was exacerbated by grout pocket corner cracks, which did not develop in the cast-in-place test. Thus, it is reasonable to use a smaller Ψ_c term to account for the more extensive cracking in the connection region of grout pockets. The use of epoxy-coated reinforcement may also contribute to earlier development of splitting cracks, although the extent of its influence on capacity is not known.

Figure 6.17 plots P_{max} vs. P_{CCD} after multiplying P_{CCD} by Ψ_c equal to 0.78. This shows a very close fit between the data and Equation 6-17. SL05 and SL06 are the only tests that produced results more than a standard deviation below the mean (as much as 23 percent lower). Interestingly, using an embedment depth of 9 in. ($9d_b$), SL05 and SL06 produced an unusually shallow cone, because of the presence of the top longitudinal bars. The failure surface for these tests was defined by spalling of the cover concrete over the top longitudinal bars. At an embedment of $12d_b$, SL03 and SL04 exhibited a much deeper cone, as the failure surface was able to form beneath the top longitudinal bars. This likely accounts for the discrepancy. Although connectors in an actual bent cap system will use deeper embedment depths, these data points are conservatively included in the calibration of the design equation.

Table 6.9 shows results for four single-line tests not included in Figure 6.17. Inclusion of SL10 in the calibration would have been too conservative because SL10 was not close to producing a breakout surface due to the limited force associated with the small #6 bar. SL16 represents the cast-in-place specimen. As discussed in Section 3.3.4.7, SL17 and SL18 achieved significantly larger forces due to the use of confining reinforcement. All other specimens used in calibrating Equation 6-17 either did not use such reinforcement or did not benefit from it. Figure 6.18 shows the relation of these four tests (shaded points) to the other test data.

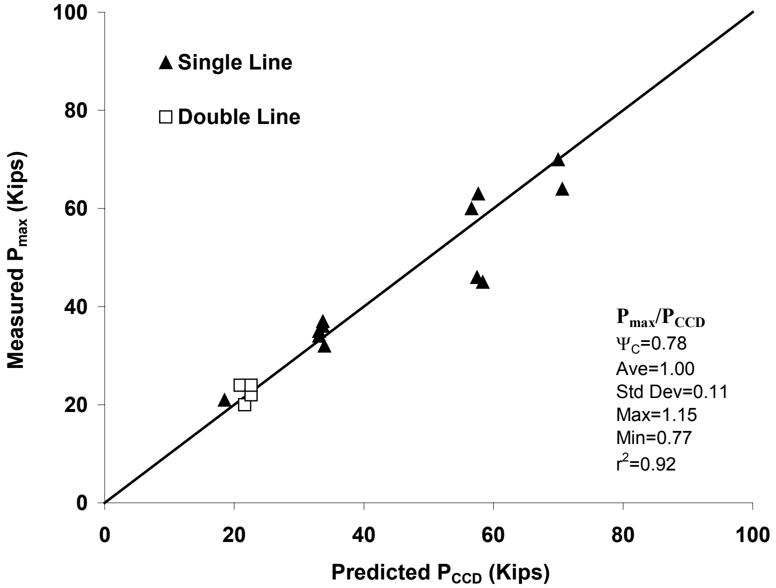


Figure 6.17 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests (Ψ=0.78)

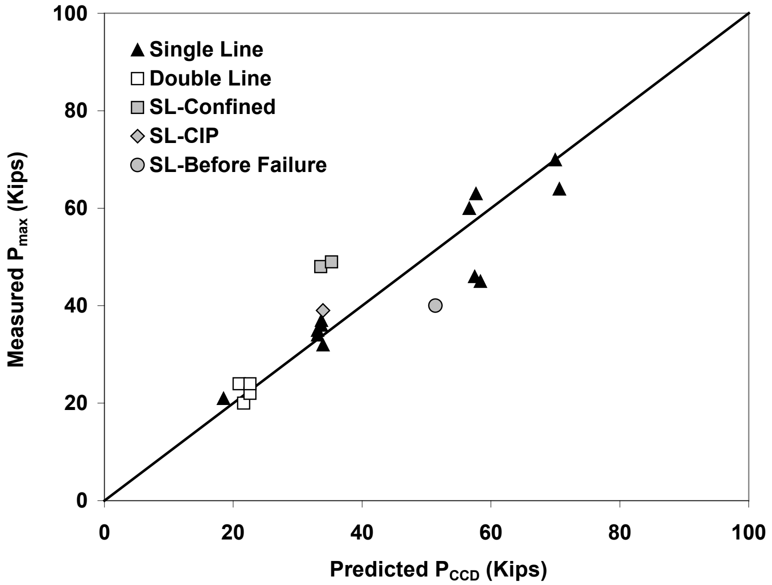


Figure 6.18 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests (All)

Table 6.9 Comparison of Predicted and Actual Capacity—Additional Phase 1 Headed Bar Grout Pocket Tests

Test	Bars	h_{ef}	f'_c (ksi)		P_{max}	A_N	A_{NO}	$A_{N/}A_{NO}$	c_{min}	$1.5h_{ef}$	Ψ_E	Ψ_C	$P_{CCD}{}^A$	P_{max}/P_{CCD}	$f'_{c1}/f'_c{}^D$	P_{max}/P_{CCD}
ID	no.-size	in	concr	grout	kips/bar	in^2	in^2		in	in			kips/bar			$(\Psi_C=0.78)$
SL10	1-#6	8	5.3	7.5	40	576	576	1.00	12	12	1.00	1	66	0.61	1.06	0.77
SL16^B	2-#8	12	5.0	-	39	1056	1296	0.81	12	18	0.90	1	43	0.90	-	1.14
SL17^C	2-#8	12	4.9	6.2	48	1056	1296	0.81	12	18	0.90	1	43	1.11	1.02	1.41
SL18^C	2-#8	12	5.4	5.6	49	1056	1296	0.81	12	18	0.90	1	45	1.08	1.00	1.37

Footnotes

A. $P_{CCD}=(A_n/A_{no}\Psi_E\Psi_C)40f'_c{}^{0.5}h_{ef}{}^{1.5}$ ($h_{ef}\leq 11$ in.)

$P_{CCD}=\max((An/Ano\Psi_E\Psi_C)40f'c^{0.5}hef^{1.5}, A_n/A_{no}\Psi_E\Psi_C)27f'_c{}^{0.5}h_{ef}{}^{1.5})$ (25 in.>h_{ef}>11 in.)

B. Cast-in-place specimen

C. Confining reinforcement enhanced capacity

D. $f'_{c1}=[f'_c(1-A_g/A_n)]+f'_{cg}(A_g/A_n)$

Table 6.8 also shows that grout strength plays a negligible role in anchorage capacity due to the relatively small area of grout intersecting the failure surface. Table 6.8 calculates the ratio of a combined compressive strength, f'_{c1} to f'_c. The value of f'_{c1} is determined from a weighted average of the grout and concrete strengths based on the area of concrete and grout within the projected concrete failure area, A_n (see Note C in Table 6.8). The average value of f'_{c1}/f'_c is 1.03, with the larger values of f'_{c1}/f'_c occurring only at very shallow embedment depths for which the grout strength is more heavily weighted. Because such shallow embedment depths are impractical for a precast bent cap system and because f'_{cg} will always exceed f'_c in design, f'_c can be conservatively used.

6.4.4.3.3 Modified CCD Design Equation

Based on the foregoing development, a modified CCD equation for the nominal concrete breakout strength for a group of fasteners in tension, P_{cbg}, is as follows:

$$P_{cbg}=\frac{A_N}{A_{No}}\Psi_E\Psi_C P_b \qquad (6\text{-}18a)$$

where Ψ_C = modification factor to account for grout pocket connection cracking

= 0.75

P_b = basic concrete breakout strength in tension of a single fastener, lbs

$$= 24\sqrt{f'_c}h_{ef}^{1.5} \qquad h_{ef}\leq 11 \text{ in.} \qquad (6\text{-}18b)$$

$$= 16\sqrt{f'_c}h_{ef}^{5/3} \qquad h_{ef}>11 \text{ in.} \qquad (6\text{-}18c)$$

This is the same as Equation 6-8 in Section 6.3.9.3.4. The differences between the terms used in Equation 6-18 and those defined for Equation 6-17 include: 1) the use of a cracking term, Ψ_C, equal to 0.75, and 2) smaller coefficients for Equations 6-18b and 6-18c. The use of Ψ_C less than 1.0 represents the influence of grout pocket cracking on breakout capacity not exhibited for cast-in-place anchors. The 0.78 value discussed in the previous section is conservatively rounded down to 0.75. The design coefficients of 24 and 16 used for P_b are based on the large database from which the CCD method was developed. These coefficients correspond to the use of a nominal strength based on the 5 percent fractile, i.e., a 90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength. In effect, this incorporates a safety factor of approximately 1.7 into Equations 6-18b and 6-18c. Group behavior is directly accounted for by the A_N/A_{No} term.

CCD equations were based on test data that included embedment depths no larger than 25 in. Although this presents some uncertainty for the deeper embedment depths expected for some bent caps, the degree of conservatism incorporated in Equation 6-18 as well as behavior of grout pocket connections in Phases 2 and 3 suggest that the use of Equation 6-18 provides a conservative basis. Other possible conservatisms include:
1) higher value of f'_c than specified, 2) use of uncoated connectors, 3) use of larger connector heads, and 4) contribution of confining reinforcement. Because of the large safety factors already included in Equations 6-18b and 6-18c, it is recommended that the design force for connectors be based on the specified yield strength of the connectors, rather than $1.25f_y$. This provides a safety factor on the order of that provided for Equations 6-13 and 6-15.

6.4.4.3.4 Design Considerations

The development of a modified CCD equation for connectors anchored in a grout pocket was based on pullout tests in which connectors were subjected to pure tension. The application of the CCD method to a precast bent cap system using grout pocket connections is complicated by the following:

1. Actual connectors in a precast bent cap system are not subjected to pure tension
2. Connections must transfer forces associated with combined transverse and longitudinal eccentricities
3. The number of connectors to be included in group strength depends on the actual load combination under consideration
4. Connectors may not be expected to yield

These issues must be considered in the application of Equation 6-18 to design.

Connectors can be designed using CCD provisions for tension alone. Although the CCD equations have been developed for combined tension and shear [6.23], they are not considered appropriate because the failure modes assumed in the development of CCD shear provisions (i.e., steel strength in shear, concrete breakout surface in shear, and concrete pryout) are not expected for normal connector configurations for a precast bent cap system and were not observed in Phase 2 and 3 tests. As mentioned in Section 6.3.8.2.2, direct shear at the cap-to-column or pile interface is designed using shear friction.

Designers should use the actual P-M combinations from analysis to determine which connectors are expected to yield in tension, then use those connectors to determine the failure surface based on group action. However, it is possible that few or none of the connectors would be expected to reach yield at the factored level, even for worst-case eccentricity combinations. In addition, based on the location of connectors with respect to the neutral axis for biaxial bending, some connectors will be highly stressed in tension whereas others may be close to the neutral axis and thus at a low level of tension. Designers must therefore use judgment in deciding which connectors will be included when using the modified CCD equations. It is recommended that, in determining the embedment depth, the designer ensure that the connectors most highly stressed in tension actually can achieve yield. Regardless of which connectors are assumed to be in tension for CCD calculations, all connectors should be embedded to the same depth in the final design.

Since designers should assume the most highly stressed connectors reach yield, it may be simpler, and sufficiently conservative, to check concrete breakout in the longitudinal and transverse directions separately. For example, if a 2/3 configuration is used, a designer might find that connector embedment can be determined by checking the longitudinal direction using one row of three connectors and checking the transverse direction using two or four connectors. Experience is necessary to indicate simple yet conservative approaches. For connections that use a small bar size and a small number of connectors (like a pile bent), designers can conduct a quick and conservative check by assuming all connectors yield in tension simultaneously.

Embedment depths should initially be based on the minimum required embedment depth according to Section 6.3.9.3.7 (3/4 of the cap depth or 18 in.). Final embedment depths based on the CCD method need not exceed that required by Equation 6-13 for straight bars in grout pockets. It is reasonable to expect that the actual development length for headed bars will not exceed that for straight bars, as research on cast-in-place headed bars has shown [6.24,6.25]. In fact, when using headed bars, designers may choose to simply use Equation 6-13 for design because it is simpler to apply than the CCD method. However, experience in applying both approaches will help the designer become familiar with the most efficient and economical approach for most situations.

6.5 SUMMARY

In this chapter, a design methodology is developed for a precast bent cap system. First, standard practice used by TxDOT to design reinforced concrete interior bents is introduced. Standard practice includes the following major assumptions: 1) pinned connections at column tops for bent cap analysis and design, 2) rigid joints for lateral analysis, 3) column design often based on a “1 in.-per-ft.” height limitation and one-percent longitudinal reinforcement for column bents and 1.5 percent for pile bents, and 4) adequate anchorage of column or pile longitudinal reinforcement in the joint region when such reinforcement is extended a length equal to the cap depth minus 6 in. for column bents and 10 to 16 in. for pile bents.

In contrast to standard practice, a design methodology for a precast bent cap system has been developed. This approach applies only to the most common multi-column and trestle pile bents and is not intended to apply to single-column bents, bents subjected to seismic or other highly dynamic loads, or bents of unusual proportions or applications. Based on test results from Phases 1-3, the design methodology includes the following:

1. An eight-step design procedure that includes: 1) selection of a trial bent configuration, 2) analysis and design of the bent cap and columns, 3) determination of connection actions, 4) selection of connection type and embedment, 5) election of a trial connector configuration, 6) analysis of connector configuration at the strength and serviceability limit states, 7) determination of connector type and embedment depth, and 8) selection of confining and auxiliary reinforcement.
2. A design philosophy that incorporates ductility, redundancy, and structural integrity into the design by the following measures: 1) conservative estimates of design actions for the cap, column, and connection, 2) application of a 1.3 factor to connection design loads, 3) conservative development length equations and minimum embedment depth requirements for grout pocket and grouted vertical duct connections, 4) minimum number and area of connectors, 5) minimum area of confining reinforcement, and 6) optional use of headed reinforcement.

This chapter also summarizes the development of design equations for connector anchorage for straight bars in grout pockets, straight bars in grouted vertical ducts, headed bars in grouted vertical ducts, and headed bars in grout pockets. Development of equations for anchorage was based primarily on Phase 1 pullout tests, although Phase 2 and 3 data was used as well. For the first three cases, development length equations were established, based on a uniform bond stress model. For headed bars in grout pockets, a modified Concrete Capacity Design approach was developed. Conservative assumptions were used throughout development, due to the paucity of test data. The appendix provides plan sheets for the first bridge designed using the design recommendations.

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CHAPTER 7:
DEVELOPMENT OF A PRECAST CONNECTION SPECIFICATION

7.1 INTRODUCTION

This chapter summarizes the development of a connection specification for a precast bent cap system, referred to as a precast connection specification. Major components of the specification are developed based on results from Phases 1-3. The following areas are addressed: 1) materials, 2) precast bent cap placement plan, 3) grouting operations, and 4) additional items.

7.2 MATERIALS

To ensure selection of a proper grout, the connection specification should include a grout specification. In addition, properties of connectors and connection hardware should be carefully specified.

7.2.1 Non-Shrink Grout

Table 7.1 is a modified version of Table 2.6, the grout specification for a precast bent cap system. Based on Phase 1-3 tests, the properties specified in Table 2.6 were modified to address mechanical, compatibility, constructability, and durability properties. The reader is referred to Chapters 2 through 5 for background information on construction and grouting issues.

7.2.1.1 Mechanical Properties

The compressive strength requirement for grout is intended to: 1) provide for transfer of forces between connectors, grout, ducts, and/or concrete, 2) provide timely strength gain for rapid construction, and 3) ensure the grout is not the weak link in the connection system. Phase 1-3 tests indicated adequate grout strength for transfer of connection forces, even for the majority of cases in which the grout strength was less than that of the surrounding concrete. All of the tested grouts were expected to satisfy the strength gain requirements of Table 2.6, although this did not always occur. Masterflow 928 (MF928) exhibited the most consistency in achieving strength gain. Although the grout specification provides a reasonable minimum requirement for strength gain, project-specific requirements may be more or less stringent. Thus, the engineer should not rely solely on the grout specification, but should specify in the plans the required minimum grout strength for beam placement and the final grout strength. The contractor, in turn, is required to select a grout that achieves the necessary strength at the critical stages.

Only in one test was the grout considered to be the weak link in the system (Phase 2, VD04, Euclid Hi-Flow [EHF]). In the VD04 pullout tests, the grout strength was just 55% of the strength of the surrounding concrete (3.1 ksi, modified grout cube strength compared to a concrete strength of 5.6 ksi). As mentioned in Chapter 1, others have reported excellent anchorage and response for grouted vertical duct connections when the grout strength was approximately equal to or as much as 1.4 ksi greater than the concrete strength [1.30,1.31].

Based on the previous discussion, it is recommended that: 1) the grout cube compressive strength satisfy the requirements of Table 7.1, and 2) the modified grout cube strength at 28-days, based on a 0.8 factor, exceed the specified 28-day concrete compressive strength by a minimum of 1000 psi. Many prepackaged grouts satisfy these two requirements. A 1000-psi margin accounts for the likelihood that the actual concrete strength will exceed the specified strength as well as the possibility of a low grout strength. The 28-day grout cube strength was increased to 5800 psi in Table 7.1 to provide a 1000-psi margin for Class C concrete.

If a bent cap uses 5000-psi concrete, then grout with a 28-day (unmodified) cube strength of at least 7500 psi is required. A number of such grouts are available. Engineers should be careful to ensure that they select a grout with a compressive strength based on the water required for fluid consistency. Grouts mixed to a flowable or plastic consistency in accordance with ASTM C 230 achieve a higher compressive strength but inadequate fluidity for grouting voids in a precast bent cap system. Manufacturers' data sheets typically list compressive strengths for all three consistencies.

7.2.1.2 Compatibility

Compatibility requirements are related to volume stability, modulus of elasticity, and coefficient of thermal expansion. Table 7.1 uses the same values as those defined in Table 2.6. The values for the modulus of elasticity and coefficient of thermal expansion provide a fairly close match for grout and the surrounding concrete. As mentioned in Chapter 2, ASTM C 1107 allows three grades of shrinkage-compensating grouts (Table 2.5): Grade A—prehardening volume-controlled type, Grade B—post-hardening volume-controlled type, and Grade C—combination volume-controlled type. MF928 is a Grade B grout, whereas EHF and Sika 212 are Grade C. Tests confirmed that for connections using Grade B and Grade C grouts, cracking did not develop in the connection region prior to loading. No deficiencies in behavior were attributed to Grade type. Thus, Table 7.1 lists either Grade B or Grade C as acceptable grout types. Grade A grouts were eliminated because they can produce as much as a 4-percent volume expansion before the grout hardens, possibly causing a reduction in density of the hardened grout, as well as larger shrinkage stresses.

7.2.1.3 Constructability

Proper grout flowability is a key to successful construction of a precast bent cap system. Table 7.1 specifies a fluid consistency for grout, with an efflux time, or flow, between 20 and 30 seconds as determined by the Flow Cone Method per CRD-C 611 and ASTM C 939. The lower limit has been changed from Table 2.6, which specified a flow between 10 and 30 seconds.

In tests that used grouts with a flow that longer than 30 seconds, the greater grout viscosity slowed down the venting of air bubbles from the grout, often resulting in an air void at the top of the pocket. This could provide a moisture path into the connection and threaten durability. On the other hand, a grout with too short of a flow time may be indicative of segregation. This was observed particularly with Sika 212 grout in Phases 2 and 3. Segregation resulted in a denser grout at the bottom of connections, but pasty, weak material near the top surface. To prevent segregation, the lower range has been increased to 20 seconds. When needed, ice or warm water may be used in grout mixing to help adjust the flow. For some temperature ranges, this will also increase the working time. No problems with set time were observed in the test program. Thus, the range was not changed from Table 2.6.

The working time, or pot life, of the grout is a crucial consideration in grout selection. Based on Phase 2 and 3 grouting, it is expected that a contractor will require approximately 15 minutes to gravity-flow grout a 30-in. deep cap with a double line grout pocket connection or a connection with four ducts. Although longer times should be estimated for deeper caps or additional ducts, grouting of an individual connection is not expected to require more than 30 minutes. Pumping of grout is expected to reduce grouting time. It is important that the estimate of the total grouting time account for: 1) conducting the flow cone test, 2) transferring grout from the mixer to dispensers, 3) transporting grout to point of placement, and 4) grouting one or more connections. Water and air temperatures at the jobsite must also be considered.

Table 7.1 Precast Bent Cap Grout Specification

Property	Values	
Mechanical	**Age**	**Compressive strength (psi)**
Compressive strength (ASTM C-109, 2” cubes)	1 day	2500
	3 days	4000
	7 days	5000
	28 days	max[5800, 1.25($f'c_{cap}$+1000)]
Compatibility		
Expansion requirements (ASTM C 827 & ASTM C 1090)	Grade B or C—expansion per ASTM C 1107	
Modulus of elasticity (ASTM C-469)	3.0-5.0×10^6 psi	
Coefficient of thermal expansion (ASTM C-531)	3.0-10.0×10^{-6}/deg F	
Constructability		
Flowability (CRD-C 621/ASTM C-939)	fluid consistency efflux time: 20-30 seconds	
Set Time (ASTM C-191)		
Initial	3-5 hrs	
Final	5-8 hrs	
Durability		
Freeze Thaw (ASTM C-666)	300 cycles, RDF 80%	
Sulfate Resistance (ASTM C-1012)	expansion at 26 weeks < 0.1%	

7.2.1.4 Durability

As mentioned in Chapter 2, grout durability should be at least equal to that of the surrounding concrete, and proprietary grouts are often formulated to achieve this. Because examination of grout durability was beyond the scope of this research, specific properties of proprietary grouts were not investigated. Requirements listed in Table 7.1 should be checked against project-specific requirements. In addition, manufacturers should be consulted for available properties, such as resistance to freeze-thaw, chlorides, sulfates, and scaling.

In some cases, specially-modified grouts such as latex-modified grouts may be useful. Such grouts cannot be recommended based on the scope of this research. However, future research may show other alternatives to be viable. Specifying durability requirements for a cementitious grout is generally expected to eliminate lesser quality grouts. However, engineers should be careful that grouts do not satisfy durability requirements at the expense of other required properties.

The following minimal provisions are recommended in selecting durable grouts: 1) grouts should be chloride-free, 2) grouts should use non-metallic formulations. Bleed properties should also be reviewed, if available.

Provisions for durability enhancement of the connection region are discussed in Chapter 6.

7.2.2 Connection Hardware

Connection hardware refers to connectors, ducts, anchor plates, shims, and other similar items required for connection construction. Connector and duct requirements are discussed in Chapter 6. Connectors should conform to the requirements of the *Materials* section of the Precast Connection Specification.

The engineer should specify in the plans any requirements for plates and other items necessary for anchorage of bolted connections. These items were not addressed in a detailed way in Phases 2 and 3.

7.2.2.1 Shims

Shims were found to be a reliable means for cap support. In Phase 2, both steel and plastic shims were effective for cap placement. Shims were glued together and to the column or pile surface to prevent movement. In Phase 3 the contractor did not glue individual shims together or glue shimpaks in place. Workers thus found steel shims to provide better stability than plastic shims.

It is recommended that both steel and plastic shims be permitted. Plastic shims should be an engineered multipolymer high-strength plastic. Specific measures to prevent movement of shims during cap placement should be detailed in the plan sheets. Prior to cap placement, the underside of the cap should be checked to ensure a flat bearing surface. Two shims may be used at exterior columns or piles to ensure bearing on at least three of the four shims. To facilitate complete grouting of the bedding layer, the total shim plan area should be limited to approximately 10% of the pile or column top area. Limiting individual shims to an aspect ratio of two may also help. Shims should be sized to ensure the allowable bearing stress at both concrete surfaces is not exceeded. In addition, shims should be placed at least 2 in. away from surface edges to help ensure grout completely surrounds shims. Additional cover may be required for corrosion protection of steel shims.

7.3 PRECAST BENT CAP PLACEMENT PLAN

To ensure the contractor uses an appropriate construction sequence and carefully plans all operations associated with cap placement, the contractor should submit a Precast Bent Cap Placement Plan to the engineer for approval prior to mixing a trail batch of grout. This plan should include: 1) a step-by-step description of the construction sequence, 2) a step-by-step description of grouting operations, 3) the method for cap support prior to and during grouting, 4) manufacturer's literature for a minimum of two candidate connection grouts, and 5) manufacturer's literature for connection hardware.

7.3.1 Construction Sequence

Example construction sequences for a precast bent cap system are discussed in Section 2.2. The contractor should completely describe the proposed construction sequence. In addition, a description of other pertinent information should be outlined, such as the method to provide anchorage holes in the piles or columns (i.e., embedded sleeves vs. drilled holes) or the use of special devices to assist in threading the cap over connectors.

7.3.2 Grouting Operations

A detailed description of grouting operations should address formwork, air venting, grouting method, and sequence of steps. These issues are discussed in Section 7.4.

7.3.3 Cap Support

The contractor should indicate the method and hardware for cap support prior to and during grouting. Hardware will likely consist of shims, friction collars, bearing plates and leveling nuts, shoring or other systems. The contractor should define the support systems and provide product information, material descriptions, and drawings, as appropriate.

7.3.4 Manufacturer's Literature for Candidate Grouts

The contractor should identify two candidate grouts for connections and provide the manufacturers' literature. Selected grouts should satisfy the grout criteria listed in Table 7.1. In addition, literature should indicate mixing requirements, working time, curing requirements, and other pertinent information. Two grouts should be selected in the event that the first grout is found unacceptable during the trial batch.

7.3.5 Manufacturer's Literature for Connection Hardware

The contractor should also provide manufacturers' literature for all connection hardware to be used.

7.4 GROUTING

The precast connection specification should include specific requirements for all grouting operations, including: 1) a trial batch, 2) formwork, 3) presoaking, 4) pre-grouting meeting, and 5) grouting methods. The following sections discuss these requirements.

7.4.1 Trial Batch

The trial batch of grout should be prepared a minimum of two to four weeks prior to connection grouting. The requirement for a trial batch is especially important because a trial batch enables contractor personnel to assess the suitability of a grout for constructability and strength, and also provides the contractor valuable experience. During Phase 3, the contractor confirmed the importance of a trial batch. As mentioned in Section 5.2.5.1, the specific purposes of a trial batch are:

1. To determine the required amount of water to be added to a particular grout brand to achieve acceptable flowability using the CRD-C 611/ASTM C 939 Flow Cone Method under the temperature conditions expected in the field
2. To determine the grout cube strength corresponding to the flow achieved
3. To examine grout for undesirable properties such as segregation
4. To establish the adequacy of proposed grouting equipment such as the mixer, tremie tubes, funnels, buckets, and vent tubes
5. To provide jobsite personnel experience in mixing and handling grout prior to connection grouting
6. To help the contractor to make a judicious decision regarding grout brand

The following sections highlight important lessons learned during Phase 1-3 grouting, which should be applied in trial batches.

7.4.1.1 Equipment

The contractor should use the proposed grouting equipment in all mixing and grouting operations. Equipment such as a mortar mixer, tremie tubes, funnels, buckets, and vent tubes should be carefully selected. The proposed mixer for actual grouting should be used for mixing trial batches. High-speed hand drills mix grout more thoroughly, but cannot produce a sufficient volume for connection grouting. The inside diameter of the tremie tube should be large enough for grouting in a timely manner, but small enough to drain the funnel volume gradually so that a continuous grout flow is maintained. In addition, the outside diameter of the tube should be small enough to fit between the duct walls and connectors. Funnels should be large enough to ensure a continuous flow of grout within the tube. A minimum funnel size of 4 quarts is recommended. A pinch valve in the tube is recommended and should be required for cases in which an interruption in grouting operations may occur. Bucket volume should be at least 5 gallons. The inner diameter of air vent tubes should be at least 0.5 in. Transparent vent tubes will accommodate visual inspection of air venting better than opaque tubes or vent holes. A 0.5-in. minimum wire mesh (hardware cloth) should be used as a filter to remove potential clumps when dispensing grout from the mixer.

7.4.1.2 Grout Flowability

A main purpose of the trial batch is to determine the required amount of water to be added to a prepackaged grout to achieve acceptable flowability in the field. The trial batch of grout should be mixed using water at a temperature corresponding to that expected for field grouting, and also at the expected ambient air temperature. This is important, as some grouts only achieve the fluidity and strength stated in the literature when mixed at an ideal temperature of 70 degrees Fahrenheit. The manufacturer's recommended amount of water to achieve fluid consistency may be used in the first batch.

After mixing in accordance with manufacturer's recommendations, the grout should be inspected for undesirable properties such as segregation or clumps. A minor amount of settlement of grout solids during mixing is acceptable, but grouts exhibiting significant segregation (e.g., clear separation between mix water and fine aggregate) should not be used. Grout segregation may lead to the formation of gaps at the bedding layer or produce cavities at the top of the cap. Gaps or cavities may threaten connection durability and/or reduce the ability of the connection to transfer forces. Clumps may result when a low-speed mortar mixer is used with a large volume of grout.

The flow time should be determined using the CRD-C 621/ASTM C 939 Flow Cone Method. When collecting grout for the flow cone test, a representative portion of the grout should be used. Grout should not be obtained by skimming the top surface of grout from a mixer, as grout tends to be more fluid at the top. A 0.5-in. mesh should be used to eliminate clumps from grout used for the flow cone test. Two flow cone tests should be conducted: one immediately after mixing and a second at the expected pot life of the grout. The second test is intended to confirm that a batch of grout will maintain a suitable flowability throughout grouting operations.

If the flow time falls outside of the 20 to 30 second range, then one of the following actions should be taken: 1) slightly change the amount of water, as long as it is still within manufacturer's recommendations, or 2) use cold or warm water to adjust the flow. If this does not produce an acceptable flow, another brand of grout should be used. In each case, a new batch of grout should be mixed. Remixing, or retempering, of grout mixtures should not be permitted, as it can change grout properties and introduce extra air into the mix. In some cases, slightly increasing the amount of water (e.g., 10%) may significantly increase the flow. However, this will also reduce the grout strength.

A TxDOT Materials representative should assist in conducting the flow cone test, and should prepare and cure a minimum of nine grout cubes for each candidate grout that achieves a suitable flow. A commercial testing laboratory approved by the engineer or a TxDOT Materials representative should test the grout cube specimens. At least two cubes should be tested at 1 day, 3 days, 7 days and 28 days.

7.4.1.3 Trial Grouting Operation

Only grouts that achieve a suitable flow should be used in a trial grouting operation. All equipment proposed for actual grouting operations should be used in a simple, mock grouting operation. Grouting operations should test the suitability of tremie tubes, funnels, vent tubes, and other equipment proposed for use. This allows the grout to be further inspected for workability, segregation and excessive bleeding.

Tamping of grout is recommended instead of vibrating. However, if a vibrator is proposed for use, it should be approved by the grout manufacturer and tested during the trial grouting operation. Care must be exercised in using vibrators because excessive agitation can entrap air in the grout.

Depending on the project requirements, grouting operations may encompass a wide range of activities, from forming and grouting a mock-up of an actual connection detail to grouting a simple box or circular form. It is left to the discretion of the engineer to judge what is reasonable and prudent. However, the trial grouting operation should closely simulate the actual field conditions, including physical constraints, temperature, etc.

7.4.1.4 Scheduling, Weather Restrictions, Admixtures

7.4.1.4.1 Scheduling of Trial Batch

It is recommended that trial batches be completed at least two weeks prior to actual grouting operations. This time is necessary to conduct the trial batch, determine grout strength and strength gain, and conduct an additional trial batch if the strength or strength gain does not satisfy the specification.

7.4.1.4.2 Weather Restrictions

Grouting should be conducted under the same limitations as casting concrete. Grouting during rainy weather may not only add water to grout mixes but may also rush workers as they conduct grouting operations. To prevent poor durability or other undesirable properties, manufacturer's recommendations for cold weather limitations should be followed. In addition, cold and warm weather practices may be necessary for flowability.

7.4.1.4.3 Admixtures

Prepackaged grouts are proprietary mixes, and thus no additives should be used in the grout. Additives may adversely affect grout properties and void manufacturer warranties.

7.4.1.5 Acceptance

Any grout conforming to the following should be acceptable for use:

1. Satisfies all of the parameters of the grout specification of Table 7.1
2. Achieves an acceptable grout flow in field conditions during the trial batch immediately after mixing and at the pot life
3. Attains compressive strength and compressive strength gain based on grout cube tests using trial batch grout
4. Possesses a working time suitable for connection grouting
5. Performs reliably in trial grouting operation
6. Possesses other properties, including durability, required for a project-specific application

7.4.2 Formwork

To ensure successful grouting, the bedding layer must be properly formed. As shown in Chapter 5, flexible fiberglass forms are readily available and may be tightly wrapped around and bolted on round columns. Wood may be used to form around the bedding of square or rectangular piles or columns. Care should be exercised to ensure forms are tight and properly sealed. Presoaking is a vital step to ensure forms are sealed. Custom-made forms may be more reliable in sealing rectangular and square sections. Formwork should accommodate air vent tubes or holes. Supplementary vents may be formed into the bent cap.

7.4.3 Presoaking

Connections should be presoaked with water for a minimum of two hours prior to grouting. Presoaking connections should be conducted for two reasons: 1) to verify tightness of forms at the bedding layer, and 2) to minimize loss of moisture from the grout into the surrounding concrete that can lead to grout shrinkage. Verification of form tightness is particularly critical to successful connection grouting. An overnight or 24-hour presoaking of the connection is preferable. Residual water left in the connection after presoaking must be drained prior to grouting. This may be accomplished with auxiliary water ports provided at the bottom of the bedding layer formwork or by vacuuming.

7.4.4 Pre-Grouting Meeting

Because of the difficulty in correcting field problems after grouting, special care and oversight should be exercised prior to and during initial grouting operations. An on-site pre-grouting meeting between the contractor and a TxDOT representative should be conducted just prior to actual grouting operations to review the details of the grouting procedure and ensure lessons learned during the trial batch are incorporated. In addition, the TxDOT representative should be available for consultation during initial grouting operations and periodically thereafter. All grouting operations should be observed by a TxDOT representative for compliance with the Precast Bent Cap Placement Plan.

7.4.5 Grouting Methods

Grout should be deposited in the connection in such a way that all voids are completely filled. Both gravity-flow and pressure grouting may achieve this objective. As mentioned in Chapter 2, gravity-flow grouting involves simpler operations overall and may be less expensive. However, additional effort may be required to ensure connection voids are completely filled. Grouting using a low pressure pump requires a higher level of skilled labor in the field, but would likely result in a connection free of voids and can expedite grouting operations, especially on large projects. Both approaches can be economical, depending on specific project constraints.

This section discusses gravity-flow grouting, which was used in Phases 1 through 3. Gravity-flow grouting should be conducted using either a bucket or tremie tube.

7.4.5.1 Bucket Approach

Placement of grout with a bucket is a viable alternative for grout pocket connections, which use relatively large openings at the cap top. As discussed in Chapters 4 and 5, five-gallon buckets are recommended for placement. After mixing the grout, buckets are filled and lifted to the cap top. The grout is poured into the pocket in lifts and tamped after each lift. A flat object such as a shovel or plywood can be used to help direct grout into the pocket with minimal agitation of the grout or air entrapment. Any grout sediments remaining at the bottom of the bucket should be removed and placed into the pocket prior to tamping.

7.4.5.2 Tremie Tube

Tremie-tube grouting should be used for grouted vertical duct and bolted connections, and may also be used for grout pocket connections. One of three variations may be used: 1) continuous-flow, 2) modified, and 3) decanting. Continuous-flow tremie-tube grouting should be conducted by lowering a flexible tube to the bottom of the bedding layer and filling the connection from the bottom upward with a continuous flow of grout. With this approach, it is crucial that grout fill the tube continuously to avoid entrapping air in the connection. This requires that a sufficient amount of grout be mixed prior to grouting and that the funnel connected to the tube have adequate capacity. A pinch valve may be used to stop the flow during grouting. This allows for refilling the funnel and tamping the voids. The tube should remain within the grout, but may be gradually withdrawn as the level of grout rises in the ducts or pockets. This approach should be used when possible, as it will likely prevent air entrapment.

The modified tremie tube and decanting approaches do not require a continuous flow of grout. The modified tremie tube approach should be used in cases where the tube cannot extend to the bottom of a connection due to small clearances or other reasons. The tremie tube should always be kept above the top of the grout, and the tube should direct the flow of grout against either a connector, sidewall, or duct. The decanting approach should be conducted by pouring grout against connectors to direct the flow to the bottom of the connection. This limits grout agitation and helps prevent air entrapment. Voids should be tamped several times during grouting.

7.4.5.3 Pressure Grouting

Pressure grouting involves pumping grout into connections under low pressure. This approach may be used for all connection types, and is required for grouted sleeve couplers. The trial batch and manufacturer's guidelines should be used to establish the pressure for grouting. To prevent entrapped air, grout should not be placed at too high a rate. Voids may be lightly tamped.

7.4.5.4 Air Venting, Plan Sheets, Grout Handling

7.4.5.4.1 Air Venting

Air should be vented at the bedding layer using a minimum of four vent tubes or holes, distributed uniformly around the perimeter of the column or pile formwork. Vent tubes or holes should be located at the top of the bedding layer. When gravity flow grouting is used, multiple grout pockets or vertical ducts should be grouted from a single pocket or from a corner duct. Vents should be plugged sequentially when a steady stream of grout flow out without air. For connections with ducts or pockets, grout will eventually flow up the ducts or pockets. After the grout level in the pocket or ducts rises near the cap top, a tremie tube should be used to top off the openings.

7.4.5.4.2 Plan Sheets

It is highly recommended that plan sheets include a step-by-step list of procedures to be followed during grouting operations. This is considered a key to successful grouting operations. The contractor involved in Phase 3 construction strongly felt that this will help ensure grouting procedures are properly conducted.

7.4.5.4.3 Grout Handling

Precautions should be taken to minimize air entrapment when pouring grout from the mixer into dispensers and when grouting connections.

7.5 OTHER ITEMS

Additional items related to the precast connection specification are discussed in this section, including recommended tolerances, grout sampling for test cubes, grout curing, post-grouting inspection, and verification of connector anchorage in columns and piles.

7.5.1 Recommended Tolerances

As discussed in Chapter 6, horizontal tolerances for grout pocket connections should be +/-1 in. in the longitudinal direction and +/-2 in. in the transverse direction. Grouted vertical duct and bolted connections using ducts should provide for a horizontal tolerance of +/-1 in. in both directions. If possible, however, the engineer should size pockets and ducts to provide tolerances of at least +/-1.5 in. in both directions. These tolerances must account for combined tolerances associated with placement of connectors in piles or columns and fabrication and placement of pockets and ducts in the bent cap. Vertical tolerances should be +/-1 in.

When specifying connections using grouted sleeve couplers, the engineer should verify the available horizontal and vertical tolerances provided by a particular coupler. Different tolerances are available for different manufacturers and for couplers housing different bar sizes. In determining the suitability of such a connection, the engineer should ensure that available tolerances are compatible with tolerances of +/-1/8 in. in the horizontal direction and +/-3/8 in. in the vertical direction for placement of the coupler within the bent cap.

To ensure adequate clearances are provided, ducts should be cast in the bent cap in such a way that a vertical orientation is achieved after setting of the bent cap. This must be carefully considered during bent cap fabrication, and may be especially critical when tight tolerances are necessary such as for grouted sleeve couplers. Cross slope can be achieved by use of variable depth pedestals.

7.5.2 Grout Sampling for Test Cubes

During grouting operations, a TxDOT representative should witness the flow cone test and preparation of a minimum of six grout cubes for each batch of grout. A commercial testing laboratory approved by a TxDOT representative should test the grout cube specimens. To verify grout strength, cubes should be tested at 1 day, 3 days, and for approval of beam setting and final strength.

For cases in which inadequate strength is indicated, additional grout cubes should be tested and the average strength calculated. The engineer should determine the course of action in the event of inadequate strength, including additional grout cube testing, a review of structural calculations and durability provisions, and grout removal and re-grouting of the connection.

7.5.3 Grout Curing

All exposed grout surfaces should be cured in accordance with manufacturer's recommendations. This will typically involve covering exposed grout with clean wet rags and maintaining moisture for a minimum of 6 hours, followed by the application of an approved membrane curing compound.

7.5.4 Post-Grouting Inspection

After grout curing and form removal, all exposed grout surfaces at the top and sides of the cap and at the bedding layer should be carefully examined. If voids appear at any surface, external sealants should be applied to prevent a moisture path into the connection. In extreme cases, epoxy injection or other measures may be recommended by the engineer. In addition, external sealants should be applied to all surfaces for which enhanced durability is required.

7.5.5 Verification of Anchorage

For specific projects, the engineer may require that a pullout test be conducted to verify the adequacy of connector installation in columns, drilled shafts or piles. The connector should be loaded to less than the yield force to limit potential damage. The number of connectors to be tested is left to the engineer's discretion. The minimum force required to demonstrate adequacy of connector installation should be shown in the plans. Adequate anchorage should be assumed when an applied load equal to 85% of the specified yield strength of the connector is applied without slippage or pullout of the connector.

CHAPTER 8: RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

8.1 INTRODUCTION

This chapter presents design recommendations and the Precast Connection Specification, based on the developments shown in Chapters 6 and 7, respectively. In addition, conceptual drawings of example details for grout pocket, grouted vertical duct, and bolted connections are provided. The appendix provides plan sheets for the first bridge designed using the design recommendations.

8.2 DESIGN RECOMMENDATIONS

The following provisions represent primary recommendations for typical multi-column and trestle pile bents and are based on tests conducted in Phases 1-3 as well as the development shown in Chapter 6. Readers are encouraged to review the Chapter 6 development. Recommendations are not intended to be comprehensive, but instead focus on differences between the design of a precast bent cap system and that of a cast-in-place bent. Many design provisions for a precast bent cap system that overlap with those for a cast-in-place bent are not included. Differences center on connection analysis and design. These design recommendations are not a substitute for judgment by the engineer.

1. Design Scope and Philosophy

1.1. Scope

1.1.1. Design recommendations included herein are applicable only to multi-column and trestle pile bents and are not intended for single-column bents, bents subjected to seismic or other highly dynamic loads, or bents of unusual proportions or applications.

1.1.2. Design recommendations should be construed as minimum design requirements and are not intended to be comprehensive, but focus on differences between design of a precast bent cap system and a cast-in-place bent.

1.2. Design Philosophy

1.2.1. Constructability, durability, and force transfer requirements shall be considered in design. Force transfer requirements shall include provisions for adequate strength, ductility, stiffness, redundancy, and structural integrity.

1.2.2. Design shall satisfy applicable limit states as specified in Reference 1 or Reference 2.

1.2.3. The fatigue and fracture limit state shall be explicitly checked in design.

1.2.4. The extreme limit state accounting for vehicle collision or ship impact need not be considered in design for cases in which vehicle barriers, dolphins, or other sacrificial devices are provided.

2. Definitions

Actions – forces or moments acting on a structure

Anchorage – a length of reinforcement, or a mechanical anchorage or hook, or combination thereof at the end of a bar needed to transfer the force carried by the bar into the concrete

Bedding Layer – the layer of grout at the interface between the bottom of a bent cap and the top of a column or pile

Bent Cap – a concrete beam of rectangular or inverted-tee cross section used to transfer actions from the superstructure to the columns or piles

Connection – the means to transfer actions between the bent cap and a column or pile

Connector – a straight or headed reinforcing bar or bolt used to join the bent cap to a column or pile

Development Length, l_d – the distance required to develop the specified strength of a reinforcing bar

Edge Distance, c – the minimum distance between the centerline of a connector and the edge of the concrete

Embedded Connection – a connection for which the bedding layer and the top of a column or pile are embedded into a void at the bottom of the bent cap

Embedment Depth, h_{ef} – the depth of connector provided beyond a critical section over which transfer of force between concrete and reinforcement may occur

Grout Pocket – a precast void that is formed in the bent cap and grouted after the bent cap is placed over connectors. Single pockets formed in the bent cap cross section are referred to as single-line grout pockets; two identical pockets, formed side-by-side, are referred to as double-line grout pockets.

Grouted Vertical Duct – corrugated steel duct that is cast into the bent cap and grouted with a group of ducts that are placed over connectors

Grouted Sleeve Coupler – proprietary coupler using a metallic sleeve that is cast into the bent cap and grouted after the bent cap is placed over the connector

Longitudinal Direction – direction perpendicular to the centerline of the bent

Precast Bent Cap System – a bridge substructure consisting of a precast bent cap and cast-in-place columns or precast trestle piles, connected together in the field using connectors, grout, and possibly connection hardware

Surface-flush Connection – a connection for which the bedding layer and column or pile are below the bent cap

Transverse Direction – direction along the centerline of the bent

3. Notation

A_N = projected concrete failure area of a fastener or group of fasteners, in^2, not to exceed nA_{No}

A_{No} = projected concrete failure area of one fastener, when not limited by edge distance or spacing, in^2

= $9h_{ef}^2$

A_s = cross sectional area of a connector, in^2

c = the minimum distance between the centerline of a connector and the edge of the concrete, in.

c_{min} = smallest of the edge distances that are less than or equal to $1.5h_{ef}$, in.

d_b = nominal diameter of bar, in.

f'_c = specified concrete compressive strength of the bent cap, psi

f_y = specified yield strength of connector, psi

h_{ef} = embedment depth, in.

l_d = development length, in.

n = number of fasteners in tension in the group

P_b = basic concrete breakout strength in tension of a single fastener, lbs

= $24\sqrt{f'_c}h_{ef}^{1.5}$ $h_{ef} \leq 11$ in.

= $16\sqrt{f'_c}h_{ef}^{5/3}$ $h_{ef} > 11$ in.

P_{cbg} = nominal concrete breakout strength in tension of a group of fasteners, lbs

P_u = maximum force of connector group, assuming connector yield, lbs

Z = maximum crack width parameter, kips/in.

α = overstrength coefficient for joint shear strength

Ψ_C = modification factor to account for grout pocket connection cracking

= 0.75

Ψ_E = modification factor to account for edge distances smaller than $1.5h_{ef}$

= 1 if $c_{min} \geq 1.5h_{ef}$

= $0.7+0.3\frac{c_{min}}{1.5h_{ef}}$ if $c_{min} < h_{ef}$

4. Design Loads

4.1. Loads and Load Combinations

Loads and load combinations shall be in accordance with the applicable version of Reference 1 or Reference 2.

4.2. Loads Factors

Load factors specified in Reference 1 or Reference 2 shall be used, except as modified in the following sections.

5. Analysis

5.1. Simplified Approach

In lieu of an accurate determination of precast connection stiffness, the following provisions may be used in determination of design actions and deflections:

5.1.1. Design actions for the bent cap shall be based on analyses assuming pinned connections between the bent cap and column or pile tops.

5.1.2. Design actions for a column or pile shall be based on the governing values of two analyses: one assuming pinned connections at the column or pile top, and the other assuming a rigid connection at the column or pile top.

5.1.3. Design actions for the connection shall be based on analyses assuming rigid connections between the bent cap and column or pile tops.

5.1.4. Design shall account for actions due to simultaneous load effects in the transverse and longitudinal directions.

5.1.5. Calculation of bent deflections in the transverse direction shall be conservatively determined. Pinned connections at the column or pile tops may be assumed.

5.2. General Approach

If connection stiffness is accurately established, then it may be used for determination of design actions for the bent cap, columns or piles, and connections as well as bent deflections. To minimize the number of iterations required for each connection configuration, conservative approaches, such as that of Section 5.1.1, may be used.

6. Material Properties

6.1. Concrete

6.1.1. Concrete used in a precast bent cap system shall be normal-weight concrete with a 28-day compressive strength of at least 3600 psi.

6.1.2. The compressive strength of concrete used in design equations shall not exceed 6000 psi, regardless of the specified compressive strength.

6.2. Steel Reinforcement, Connectors and Bolts

No provisions for reinforcing steel or stands beyond those of Reference 1 or 2 are required, except as noted below:

6.2.1. Connectors other than bolts shall conform to ASTM A615 or A706. The specified yield strength shall not exceed 75 ksi.

6.2.2. Plain bars or plain wire may be used for spirals and wire fabric.

6.2.3. Bolts shall conform to ASTM A615, A193 B7, or ASTM A722. The specified yield strength of bolts shall not exceed 150 ksi.

6.3. Ducts

Semi-rigid, spirally-crimped corrugated ducts shall be used. Ducts shall be galvanized, cold-rolled steel conforming to ASTM A527 and ASTM A619. Duct thickness shall be at least 26 gage for duct diameters up to 4.5 in. and at least 24 gage for duct diameters exceeding 4.5 in. Corrugation height of ducts shall be at least 0.094 in.

6.4. Grout

Grout used in connections and in columns or piles shall conform to the requirements of the Precast Connection Specification.

7. Connection Types

Connections shall be one of the following four types: 1) grout pocket, 2) grouted vertical duct, 3) bolted, or 4) grouted sleeve coupler. Consideration should be given to the advantages and disadvantages of each connection type, particularly with respect to constructability and durability. The following provisions shall be followed:

7.1. Grout Pocket Connections

7.1.1. In the absence of other requirements, grout pockets shall be sized to satisfy the following minimum tolerances with respect to connector placement: +/-1 in. in the longitudinal direction, +/-2 in. in the transverse direction, and +/-1 in. in the vertical direction.

7.1.2. Pockets shall be trapezoidal in cross section and have flat sides, unless special circumstances dictate otherwise. Pocket taper shall be at least two degrees, in both the longitudinal and transverse directions. Taper should be minimized through the cap cross section. Roughening of pocket surfaces is optional.

7.1.3. For grout pocket connections used in column bents, double-line grout pockets shall be used. Double-line or single-line grout pockets may be used for trestle pile bents.

7.2. Grouted Vertical Duct and Bolted Connections

7.2.1. In the absence of other requirements, ducts shall be sized to satisfy the following minimum tolerances: +/-1 in. in the longitudinal and transverse directions, and +/-1 in. in the vertical direction. Duct diameter shall not exceed 5 in.

7.2.2. For grouted vertical duct connections, ducts may extend either partially or completely through the cap depth. Duct length shall exceed the embedment depth by at least 3 in. For bolted connections, ducts shall extend through the depth of the cap.

7.2.3. Ducts shall be cast in the bent cap so that vertical orientation is achieved after setting the cap.

7.2.4. Plan sheets shall include notes for anchorage hardware.

7.3. Grouted Sleeve Couplers

7.3.1. At least two manufacturers of grouted sleeve couplers shall be contacted prior to the use of a grouted sleeve coupler connection to verify available horizontal and vertical tolerances and the required construction sequence.

7.3.2. Plan sheets shall include notes of available products, required horizontal and vertical tolerances, and the recommended construction sequence.

8. Connection Design for Strength Limit State

8.1. Additional Load Factor

Factored connection actions determined from analysis shall be increased by an additional factor of 1.3 for design.

8.2. Flexure and Axial Force Effects

8.2.1. Nominal resistance of the connection shall be based on conditions of equilibrium and strain compatibility. Resistance factors shall correspond to those required by Reference 1 or 2, except where noted otherwise in the Design Recommendations.

8.2.2. Tensile strength of concrete and grout at the bedding layer shall be neglected.

8.3. Shear Friction

Design for shear friction shall be in accordance with Section 5.8.4 of Reference 2. The following shall also be satisfied:

8.3.1. If the bedding layer is not roughened to an amplitude of at least 0.25 in., the cohesion factor shall be taken as 0.075 ksi, and the friction coefficient shall be taken as 0.6.

8.3.2. If the bedding layer is intentionally roughened to an amplitude of at least 0.25 in., the cohesion factor shall be taken as 0.100 ksi and the friction coefficient shall be taken as 1.0.

8.4. Joint Shear

Joint shear design shall be in accordance with Reference 3. The following shall also be satisfied:

8.4.1. Connections shall be designed as Type I joints with an overstrength factor, α, of 1.0.

8.4.2. Vertical stirrups shall be designed through the joint region for wide bent caps in which the top longitudinal reinforcement passes outside the joint core.

9. Connection Design for Serviceability Limit State

9.1. Opening at Bedding Layer

For connections in which the column or pile is not embedded into the bent cap and protective coatings at the bedding layer are not used, potential opening between the cap and column at the bedding layer shall be determined using the following provisions:

9.1.1. Determination of potential opening shall be based on the location of the neutral axis with respect to connectors. Opening due to combined service-level flexure and axial force effects shall be investigated.

9.1.2. Duration of potential opening shall be considered.

9.1.3. If analysis indicates that a connector is subjected to tension at service load such that durability protection is expected to be compromised, one or more of the applicable provisions for durability enhancement listed in Section 13 shall be used. For cases in which potential opening at the bedding layer is expected to be negligible, use of durability-enhancement measures are optional.

9.2. Cracking at Cap Top

Crack control for the connection region shall be at least equal to that of the surrounding concrete at the top of the bent cap. Design shall be in accordance with the following provisions:

9.2.1. Grout pocket crack widths shall be assumed equal to that of the surrounding concrete at the top of the bent cap. Grout at the top surface of grouted ducts may be assumed to remain uncracked at service level.

9.2.2. Design for crack control shall satisfy the requirements of Reference 1 and Section 10.6.4 of Reference 4.

9.2.3. The maximum crack width parameter, Z, shall not exceed 130 kips/in. For other than normal exposure conditions, Z shall be based on the provisions of Reference 5 or project-specific requirements.

9.3. Deflections

For cases in which lateral deflections in the transverse direction are determined to be excessive based on the assumption of pinned connections at column or pile tops, deflections can be estimated from an interpolation of deflection values based on two bent analyses: one assuming pinned connections and the other assuming rigid connections. Engineering judgment should be used to assess the degree of fixity that exists between the bent cap and columns or piles.

10. Connection Embedment

Connections shall be designed as either surface-flush or embedded. Durability and constructability shall be considered in determining connection embedment.

10.1. Surface-flush

For surface-flush connections, the specified thickness of the bedding layer shall not be less than 1.5 in. and not more than 4 in.

10.2. Embedded

10.2.1. For embedded connections, the specified embedment into the cap, measured from the bottom surface of the cap, shall not be less than 2 in. and not more than 5 in.

10.2.2. Plans for inspection during and after grouting shall be established and documented prior to grouting.

11. Connection Detailing

11.1. Connector Type

Connectors shall be one of the following types: 1) straight reinforcing bars, 2) headed reinforcing bars, 3) hooked reinforcing bars, or 4) bolts.

11.1.1. All connector material shall conform to Section 6.2.

11.1.2. Headed bars shall be used in grout pocket or grouted vertical duct connections for bents in which reserve anchorage strength is desired or for bents subjected to large (non-seismic) lateral loads.

11.2. Limitations on Connector Size, Area, and Number

Connector size, area, and number shall be in accordance with the following provisions:

11.2.1. When standard ASTM reinforcing bars are used, bar sizes shall be no smaller than #7 and no larger than #11. Bolt diameters shall be no smaller than 7/8 in. and no larger than 1.5 in.

11.2.2. The area of connector steel shall not be less than 0.7 percent of the gross cross-sectional area of the column or less than 1.0 percent of the gross cross-sectional area of the pile. Minimum required area should not be taken as additive to strength design requirements. The area of connector steel should not exceed two percent unless constructability is carefully considered, including size and spacing of connectors, ducts, and bent cap reinforcement.

11.2.3. The following minimum number of connectors shall be used for a connection: four connectors for column bents and three connectors for trestle pile bents. Connectors shall be distributed over the column or pile cross section.

11.3. Connector Spacing

Connectors shall be spaced in accordance with the following provisions:

11.3.1. Clear spacing between connectors, including heads, shall not be smaller than the nominal diameter of the connector or 1 in.

11.3.2. For clear spacing between straight bars in grout pockets smaller than two times the nominal diameter of the connector, the development length required by Equation 12.2-1 shall be multiplied by a factor of 1.5.

11.4. Duct Spacing

Ducts in the cap and column or pile shall be spaced in accordance with the following provisions:

11.4.1. Clear spacing between ducts shall not be less than 1.5 in. or 4/3 times the maximum coarse aggregate size in the concrete.

11.4.2. For clear spacing between ducts smaller than one duct diameter, the development length required by Equation 12.3-1 for grouted vertical duct connections shall be multiplied by a factor of 1.5.

11.4.3. For all connection types, for clear cover over ducts in columns or piles smaller than 3 in., the development length required by Equation 12.3-1 shall be multiplied by a factor of 1.5.

11.5. Grouted Sleeve Coupler Spacing

Spacing for grouted sleeve couplers shall satisfy the provisions of Section 11.4 and manufacturer's recommendations.

11.6. Cover

11.6.1. Side and top cover for connectors in grout pocket connections shall be no less than 3 in. Cover shall be determined as the distance from the connector to the respective face of the bent cap.

11.6.2. Minimum concrete cover for steel reinforcement shall be in accordance with Reference 1 or 2.

11.6.3. Minimum concrete cover for connection hardware shall be the same as that for reinforcing steel.

11.7. Confining Reinforcement

11.7.1. Transverse confining reinforcement, consisting of spirals or closed ties, shall enclose grout pockets and vertical ducts used in connections.

11.7.2. Minimum confining reinforcement shall satisfy the requirements of Section 11.5.5.3 of Reference 4.

11.7.3. Confining reinforcement shall extend the full height of ducts and grout pockets or between the top and bottom layers of longitudinal reinforcement in the bent cap, regardless of connector embedment depth.

11.7.4. Vertical center-to-center spacing of transverse reinforcement shall not be less than 2 in. nor greater than 6 in.

11.8. Auxiliary Reinforcement

Auxiliary reinforcement, including anchorage hardware, shall be designed for bolted connections. Proprietary and non-proprietary hardware may be used.

12. Connector Development Length

12.1. General

12.1.1. For grout pocket and grouted vertical duct connections, the development length shall be based on the applicable provisions of Sections 12.2 through 12.5. The connector embedment depth shall not be less than the development length and shall account for required vertical tolerances.

12.1.2. Bolts used in bolted connections shall extend through the entire cap depth. Embedment of connectors used in grouted sleeve couplers shall satisfy the manufacturer's recommendations.

12.1.3. The critical section for the connector embedment depth shall be taken as the top of the bedding layer for development into the bent cap and the bottom of the bedding layer for development into columns or piles. Connector heads shall not be included as part of the embedment depth.

12.1.4. Development lengths determined using equations in Sections 12.2 through 12.5 shall apply to both coated and uncoated connectors.

12.1.5. All connection types shall satisfy the provisions of Section 12.7 for development length in columns or piles.

12.2. Straight Bar in Grout Pocket

The required development length for a straight reinforcing bar embedded in a grout pocket connection shall be:

$$l_d = \frac{0.022 d_b f_y}{\sqrt{f'_c}} \tag{12.2-1}$$

12.3. Straight or Headed Bar in Grouted Vertical Duct

The required development length for a straight or headed reinforcing bar embedded in a grouted vertical duct connection shall be:

$$l_d = \frac{0.024 d_b f_y}{\sqrt{f'_c}} \tag{12.3-1}$$

12.4. Headed Bar in Grout Pocket

The embedment depth, h_{ef}, for a headed reinforcing bar used in a grout pocket connection shall not be less than the smaller of the development length of Equation 12.2-1 and that which satisfies the following provisions:

$$P_u \leq \phi P_n$$

where:

$$P_u = 1.25 n A_s f_y$$

$$\phi = 0.90$$

$$P_n = \frac{A_N}{A_{No}} \Psi_E \Psi_C P_b \tag{12.4-1}$$

$$P_b = 24\sqrt{f'_c}\, h_{ef}^{1.5} \qquad h_{ef} \leq 11 \text{ in.} \tag{12.4-2}$$

$$P_b = 16\sqrt{f'_c}\, h_{ef}^{5/3} \qquad h_{ef} > 11 \text{ in.} \tag{12.4-3}$$

12.5. Hooked Bar in Grout Pocket

The development length for a hooked reinforcing bar used in a grout pocket connection shall not be less than that required by Equation 12.2-1.

12.6. Limitations on Embedment Depth

12.6.1. The embedment depth for connectors in grout pocket or grouted vertical duct connections shall not be less than 18 in. or three-quarters of the cap depth.

12.6.2. Top cover for connectors shall satisfy Section 11.6.2.

12.7. Straight or Headed Bars in Columns or Piles

12.7.1. When a cementitious grout is used for connector anchorage in accordance with the Precast Connection Specification, development length for a straight or headed reinforcing bar embedded in a column or pile shall be taken as that required by Section 12.5.1 of Reference 4 for a Class B tension splice.

12.7.2. When epoxy grouts or other non-cementitious grouts are used for connector anchorage, development lengths shall be in accordance with manufacturer's recommendations. Development lengths shall be adequate to develop 1.25 times the specified yield strength of the connector.

13. Durability Enhancement

The connection shall be designed to provide corrosion protection of the connectors and exposed connection hardware throughout the life of the structure. For connections in which a greater measure of durability is required, one or more of the following approaches shall be used:

13.1. Protective Coatings

Epoxy coating or galvanizing of connectors and connection hardware may be used to provide corrosion protection of connectors and connection hardware. Stainless steel may also be used. Coatings shall conform to the requirements of Reference 1 or 2.

13.2. Connection Embedment

The column or pile may be embedded into the bent cap to provide corrosion protection of connectors. Connection embedment shall be in accordance with Section 10.2.

13.3. Height of Pockets and Ducts

Height of pockets and ducts used in grout pocket and grouted vertical duct connections may be limited to less than the full cap depth.

13.4. External Sealants

External sealants may be used to provide corrosion protection of connectors and to prevent disintegration of the grout.

13.4.1. Non-porous sealants shall be applied to all exposed grout surfaces.

13.4.2. Scheduled inspection and maintenance of sealants should be indicated in design documents.

13.5. Other Measures

13.5.1. Continuous drip beads may be provided along the underside of the bent cap to limit moisture at exposed portions of the bedding layer.

13.5.2. Encasements or cap top recesses may be provided to prevent corrosion of exposed bolted connection hardware.

13.5.3. Bolts used in connections may be post-tensioned to prevent or limit potential gap opening at the bedding layer. Anchorage zones for post-tensioned connections shall be properly reinforced.

13.5.4. Waterstops may be used in the bedding layer to inhibit moisture penetration.

14. References

1. American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, 16th ed., AASHTO, Washington, D.C., 1996.

2. American Association of State Highway and Transportation Officials (AASHTO), *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units*, 2nd ed., AASHTO, Washington, D.C., 1998.

3. ACI-ASCE Committee 352, “Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-91),” American Concrete Institute, Farmington Hills, Mich., 1991.

4. ACI Committee 318, “Building Code Requirements for Structural Concrete and Commentary,” *ACI 318-99/ACI 318R-99*, American Concrete Institute, Farmington Hills, Mich., 1999.

5. ACI Committee 224, “Control of Cracking in Concrete Structures (ACI 224R-90),” American Concrete Institute, Farmington Hills, Mich., 1990.

6. ACI Committee 318-B, “Fastening to Concrete (Code CB-30),” American Concrete Institute, Detroit, 1998.

7. Matsumoto, Eric E., “Development of a Precast Bent Cap System,” Ph.D. Dissertation, The University of Texas at Austin, 2000.

8.3 PRECAST CONNECTION SPECIFICATION

The following Precast Connection Specification is based on extensive collaboration with Mr. John P. Vogel of the Houston District of the Texas Department of Transportation (Bridge Design Section), who authored the first draft. The format essentially conforms to Texas Standard Specifications [8.1].

The specification addresses many of the issues discussed in Chapter 7, which readers are encouraged to review. An attempt was made to limit excessive prescriptions that may constrain the contractor’s innovation and efficiency. However, a significant level of detail was deemed necessary for some provisions, based on important lessons learned from construction of Phase 1-3 specimens. It is expected that this document will evolve as more field experience is obtained. This specification is not a substitute for the engineer’s judgment based on design and field experience.

XXXX Specifications

SPECIAL SPECIFICATION

ITEM XXXX

PRECAST CONNECTIONS

XXXX.1. Description. This item shall govern for connection of precast concrete bent caps to cast-in-place columns, drilled shafts and prestressed concrete piles.

XXXX.2. Materials. All materials shall conform to the pertinent requirements of the following Items except as otherwise required herein:

Item 420, “Concrete Structures”
Item 421, “Portland Cement Concrete”
Item 425, “Prestressed Concrete Structural Members
Item 435, “Elastomeric Materials”
Item 440, “Reinforcing Steel”
Item 442, “Metal for Structures”
Item 447, “Structural Bolting”
Item 449, “Anchor Bolts”

(1) Hydraulic Cement Grout (Non-Shrink)

All grout for precast connections shall consist of prepackaged, cementitious, non-shrink grout in accordance with ASTM C-1107 and the additional performance requirements listed in Table 1, including mechanical properties, compatibility, constructability, and durability. Table 1 requirements shall govern over ASTM C-1107 requirements. Grout using metallic formulations will not be allowed. Grout shall be free of chlorides. No additives shall be added to prepackaged grout.

(2) Connection Hardware

All connection hardware, connectors, and ducts shall be in accordance with the requirements shown in the plans.

XXXX.3. Contractor Submittals. At least one month prior to the start of precast bent cap placement, the Contractor shall submit to the Engineer a Precast Bent Cap Placement Plan. Caps shall not be set until the Engineer has approved all required submittals. At a minimum, the plan shall contain the following items:

a) Step-by-step description of bent cap placement for each bent, including proposed method to form the connection and ensure grout is properly consolidated in the connection and bedding layer.

b) Method and description of hardware used to hold bent cap in position prior to connection grouting. Hardware may consist of plastic shims, steel shims, friction collars, shoring or other support systems. Total shim area for each connection shall not exceed 10% of the cross sectional area of the column, drilled shaft or pile. Individual shims shall be limited to a ratio of length to width of 2:1. Hardware submittal shall consist of product information for plastic shims and friction collars, drawings and material description for steel shims, and shop drawings for shoring if used.

c) Method of installing connectors. Manufacturer's literature for connector hardware and adhesives used to install the connectors in the columns, drilled shafts or piles. Literature shall include step-by-step installation instructions for adhesives used to install connectors and material properties of the adhesive. Connector hardware shall conform to the type, coating, and installation requirements shown in the plans. Submittals for connectors shall include design calculations showing that the embedment depth exceeds that required to yield connectors. Installation of anchors shall be in accordance with Item 420.11 (9) "Installation of Dowels and Anchor Bolts".

d) Manufacturer's product information for two candidate grouts, to include a description of the performance characteristics as specified in Table 1, mixing requirements, working time, curing requirements, and other information related to grouting of precast connections utilizing ducts or grout pockets.

e) Other required submittals shown on the plans or requested by the Engineer relating to successful installation of precast bent caps and associated hardware.

XXXX.4. Construction Methods.

(1) General.

The Contractor shall follow the Precast Bent Cap Placement Plan, including all manufacturer's recommendations for anchorage installation and grouting operations. At the request of the Engineer, a pre-grouting meeting shall be held to review grouting procedures.

When grout pocket connections are used, tolerance for placement of columns, drilled shafts and piles shall be +/-1 in. in the longitudinal direction and +/-2 in. in the transverse direction. Horizontal tolerances shall be taken with respect to the centerline of the bridge. When connectors are embedded in ducts, tolerance for placement of columns, drilled shafts and piles shall be +/-1 in. Size, type, location and orientation of ducts to account for cap slope shall be as shown in the plans. When connectors are installed in preexisting columns, drilled shafts or piles, the tolerance for connector placement shall be +/-1/4 in. with respect to plan location. All connectors shall be installed plumb. Vertical tolerance for cap placement shall be +/-1 inch. Tolerances for grouted sleeve couplers, if used, shall be as shown in the plans. Out-of-tolerance substructure elements shall be subject to structural review by the Engineer.

All form release agents and curing membranes shall be completely removed from areas of the cap that will be in contact with bearing seat and connection grout.

(2) Cap Placement.

The Contractor is solely responsible for insuring the stability of the bent cap prior to and during grouting operations.

All grades, dimensions and elevations shall be verified and/or determined before the bent cap is placed. The contractor shall verify proper alignment between the columns, drilled shafts or piles, including connectors, grout pockets, post-tensioning ducts, and other connection hardware cast into the bent cap. The precast cap may be set and used as a template for drilling anchorage holes at the Contractor's option.

All loose material, dirt and foreign matter shall be removed from the tops of columns, drilled shafts or piles before the cap is set.

(3) Anchorage.

A pullout test shall be used to verify the adequacy of the grout or adhesives used to anchor connectors into columns, drilled shafts or piles. The minimum force required to demonstrate adequacy of anchor installation shall be 85% of the nominal force required to yield the connector.

(4) Grouting of Connections.

Grout shall be used in strict accordance with manufacturer's recommendations.

Admixtures, including retarders, shall not be added to grout, but the temperature of mixing water may be adjusted or ice may be added to increase working time and pot life.

Addition of water to previously mixed grout or remixing of grout shall not be permitted. Water exceeding manufacturer's recommendations shall not be added to the grout to increase flowability.

(a) <u>Trial Batch</u> At least two weeks prior to grouting of connections, a trial batch of grout shall be prepared to demonstrate grout properties and adequacy of equipment and to familiarize job site personnel with grouting procedures.

A batch of grout is the amount of grout sufficient to complete an entire connection or number of connections and is limited to the amount of grout that can be placed within the pot life determined in the trial batch. Partial batches will not be allowed and shall be discarded. For continuous placement using a grout pump, a batch shall be defined as one connection or one bent cap.

The Contractor shall establish grout flowability by measuring efflux (flow) time of with a standard flow cone according to the Corps of Engineers Flow Cone Method, CRD-C 611 and ASTM C 939.

Test flow shall be determined immediately after mixing and at the expected working time to establish pot life. The ambient temperature and mixing water temperature at the time of trial batch mixing shall be the same as that expected at the time of grout placement. The Contractor shall establish that the grout flow time satisfies the limits prescribed in Table 1.

Observation of segregation or large clumps of grout in the final trial batch shall be cause for rejection of the proposed brand of grout. Samples used for testing shall be taken from the middle of the batch.

One set of six (6) grout cubes shall be prepared as specified under Section 4 (c), Grout Testing, to verify the compressive strengths shown in Table 1.

The Contractor shall validate the proposed grout placement technique by using the trial batch grout and grout equipment in a sample grouting operation similar to the proposed connection grouting. Adequacy of mixer, pump, tremie tubes, funnels, buckets, and vent tubes shall be established. The contractor shall demonstrate that the equipment provided for grouting is adequate for mixing the grout and grouting the connection within the pot life of the batch and does not introduce air into the grout or connection. A square mesh with an opening no larger than 0.5 in. shall be used to filter out potential clumps when transferring grout from the mixer to buckets.

(b) Grout Placement

Tremie tubes shall be small enough to enable grout to be placed between the connectors and corrugated ducts. Funnels shall be large enough to keep the tremie tube full at all times. The tremie tube shall be equipped with a pinch valve to stop flow in the event that grouting is interrupted.

All equipment necessary to properly perform grouting operations shall be present before actual grouting operations begin. All grouting operations shall be performed in the presence of the Engineer in accordance with the Precast Bent Cap Placement Plan. Grouting operations shall be performed under the same weather limitations as cast-in-place concrete and as required by the manufacturer. Grout pumping shall be required for connections that cannot be completed using buckets within the pot life established for the grout during the trial batch.

Forms shall be drawn tight against the existing concrete and sealed water tight to avoid grout loss or offsets at the joint. The connection shall be presoaked with water for a minimum of two hours prior to grouting. After presoaking, the connection shall be drained of all water just prior to placement of grout.

Forms for the closure pour between the cap and column shall be adequately vented to allow air to escape during grouting. Vent tubes shall have a minimum ½-in. inner diameter and shall be flush with the top of the bedding layer. Vents shall not be plugged until a steady stream of grout flows out.

Grout shall be deposited such that all voids are completely filled. Grout shall be consolidated at intervals during placement operations for all connection types. Vibrators shall not be used. All connections shall be grouted in a manner that deposits grout from the bedding layer or bottom of the connection upward. Grout shall be placed through connection ducts and/or grout ports located at the top or side of the precast cap. When grout pocket connections are used, grout may also be deposited against the side of the pocket. When insufficient pressure is available to completely fill the duct from the bottom up, the final portion of grout may be placed from the cap top.

The Contractor shall validate the proposed grout placement technique by using the trial batch grout and grout equipment in a simple grouting operation. Adequacy of tremie tubes, funnels, buckets, and vent tubes shall be established.

All equipment necessary to properly perform grouting operations shall be present before actual grouting operations begin. All grouting operations shall be performed in the presence of the Engineer in accordance with the Precast Bent Cap Placement Plan.

Forms shall be drawn tight against the existing concrete to avoid grout loss or offsets at the joint. All previously hardened concrete surfaces that will be in contact with the grout shall be pre-watered to a surface-saturated moist condition when the grout is placed. Drain ports or holes shall be provided to allow residual water from prewatering to drain prior to grouting. Forms for the closure pour between the cap and column shall be adequately vented to allow air to escape during grouting.

Grout shall be deposited such that all voids are completely filled. Grout shall be consolidated at intervals during placement operations for all connection types. All connections shall be grouted in a manner that deposits the grout from the bedding layer or bottom of connection upward. Grout shall be placed through connection ducts and/or grout ports located at the top or side of the precast cap. When insufficient pressure is available to completely fill the duct from the bottom up, the final portion of grout may be placed from the cap top. In such cases, care shall be taken to prevent introducing air into the previously placed grout.

Care shall be taken to prevent introducing air into previously placed grout by monitoring tremie tube placement, grout flow, and rate of pour.

All exposed grout surfaces shall be cured in accordance with manufacturer's recommendations.

(c) Grout Testing. The compressive strength of the grout for “Beam Setting Strength” and “Final Strength” shall be determined using grout cubes prepared and tested in accordance with ASTM C-109. The contractor will prepare a minimum of six (6) cubes per batch. A Commercial Testing Laboratory approved by the Engineer shall test the specimens for “Beam Setting Strength” and “Final Strength.” Grout failing to meet the minimum required compressive strength may be cause for rejection of the connection, grout removal, and re-grouting of the connection by means approved by the Engineer.

(5) Beam Placement.

Bearing seat build-ups, when required, shall be placed in accordance with Item 420.18. The top surface of the precast cap anchorage shall be finished in accordance with Item 420.18 or waterproofed as shown in the plans. Lifting loops shall be burned off 1 in. below the surface of surrounding concrete and patched using anchorage grout, bearing seat buildup grout or other material approved by the Engineer.

Beams shall not be set until the connection grout has reached a compressive strength equal to the “Beam Setting Strength” shown on the plans. Final acceptance of the connection shall be after the grout has reached the “Final Strength” shown in the plans and after the connection has been waterproofed, if required.

XXXX.5. Measurement. Precast connections of the type specified shall be measured by each precast connection.

XXXX.6. Payment. The work performed and materials furnished in accordance with this Item and measured as provided under “Measurement” shall be paid for at the unit price bid for each precast connection of the type specified. This price shall be full compensation for furnishing hardware to support the bent cap prior to grouting; for installation of the precast connection anchorage devices; for furnishing and mixing grout; for placing, finishing and curing the grout; waterproofing the connection; and for all labor, tools, equipment and incidentals necessary to complete the work.

Table 1: Grout Performance Specification

Property	Values	
Mechanical	**Age**	**Compressive strength (psi)**
Compressive strength (ASTM C-109, 2” cubes)	1 day	2500
	3 days	4000
	7 days	5000
	28 days	5,800 and $1.25(f'c_{cap} + 1000)$
Compatibility		
Expansion requirements (ASTM C 827 & ASTM C 1090)	Grade B or C—expansion per ASTM C 1107	
Modulus of elasticity (ASTM C-469)	$3.0\text{-}5.0 \times 10^{6}$ psi	
Coefficient of thermal expansion (ASTM C-531)	$3.0\text{-}10.0 \times 10^{-6}$/deg F	
Constructability		
Flowability (ASTM C-939; CRD-C 611 Flow Cone)	fluid consistency efflux time: 20-30 seconds	
Set Time (ASTM C-191)		
Initial	3-5 hrs	
Final	5-8 hrs	
Durability		
Freeze Thaw (ASTM C-666)	300 cycles, RDF 90%	
Sulfate Resistance (ASTM C-1012)	expansion at 26 weeks < 0.1%	

8.4 EXAMPLE CONNECTION DETAILS

This section presents conceptual drawings of example details for grout pocket, grouted vertical duct, and bolted connections. Details are shown for both trestle pile and column bents. Grouted sleeve coupler details would be similar to those shown in Chapters 1 and 2. The following details are conceptual drawings that are not intended to be standard details. All variations cannot be shown, as the design methodology is flexible and allows for many connection types and configurations. However, the details shown are representative of details that conform to the design and construction recommendations given in Sections 8.2 and 8.3 and highlight the main connection features. Actual connection details should be based on project-specific requirements and the application of provisions given in Sections 8.2 and 8.3. Drawings were prepared with the assistance of the Bridge Design Section of the Texas Department of Transportation (Austin and Houston offices).

8.4.1 Grout Pocket Connections

Figures 8.1 and 8.2 show plan, elevation, and section views of single-line and double-line grout pocket connections, respectively. Figure 8.1 shows a single-line grout pocket connection using the embedded pile option. A single line of three connectors is shown, using the minimum allowable embedment depth for connectors. Straight bars are shown, although headed bars may also be used. The actual size of the grout pocket and number of connectors would depend on project-specific requirements. Two steel shims are shown for cap setting. Confining reinforcement is also shown through the depth of the cap. The section view shows an optional air vent that is intended to complement the required air vents at the bedding layer. Corrugated ducts in the pile or column are used for connectors and are grouted separately from the grout pocket and bedding layer. Figure 8.2 shows a double-line grout pocket connection using a surface-flush option on a column bent. The column may alternatively be embedded. A double line of four connectors is shown, corresponding to a case with a large expected moment transfer. Top bars are placed between the grout pockets to minimize spacing. A double-line grout pocket is also viable for a pile bent.

8.4.2 Grouted Vertical Duct Connections

Figures 8.3 and 8.4 show details for grouted vertical duct connections, both using two rows of two connectors. Figure 8.3 shows a grouted vertical duct connection using the embedded pile option, whereas Figure 8.4 shows the surface-flush option. Both embedment options are viable for pile and column bents. Various details such as confining reinforcement, shims, and an air vent are similar to that shown for grout pocket connections. The actual size of ducts as well as the number of ducts and connectors depend on project-specific requirements.

8.4.3 Bolted Connections

Figures 8.5 and 8.6 show details for bolted connections using two rows of two connectors and a surface-flush option. Figure 8.5 shows a bolted connection with the cap set on shims, whereas Figure 8.6 shows a plate-and-leveling nut option. Figure 8.5 shows many details similar to the grouted vertical duct connection of Figure 8.4, except that the bolts extend through the entire depth of the cap and are anchored at the top of the cap with plate and nut anchorages. Figure 8.6 includes a grout port so that the bedding layer may be grouted prior to the ducts. A thicker bedding layer is shown to accommodate the leveling nut. An embedded column or pile option is viable for bolted connections in which the cap is set on shims or friction collars, but not for the plate-and-leveling nut option if field adjustment is anticipated. As for other connection types, the size of ducts as well as the number of ducts and bolts depend on project-specific requirements.

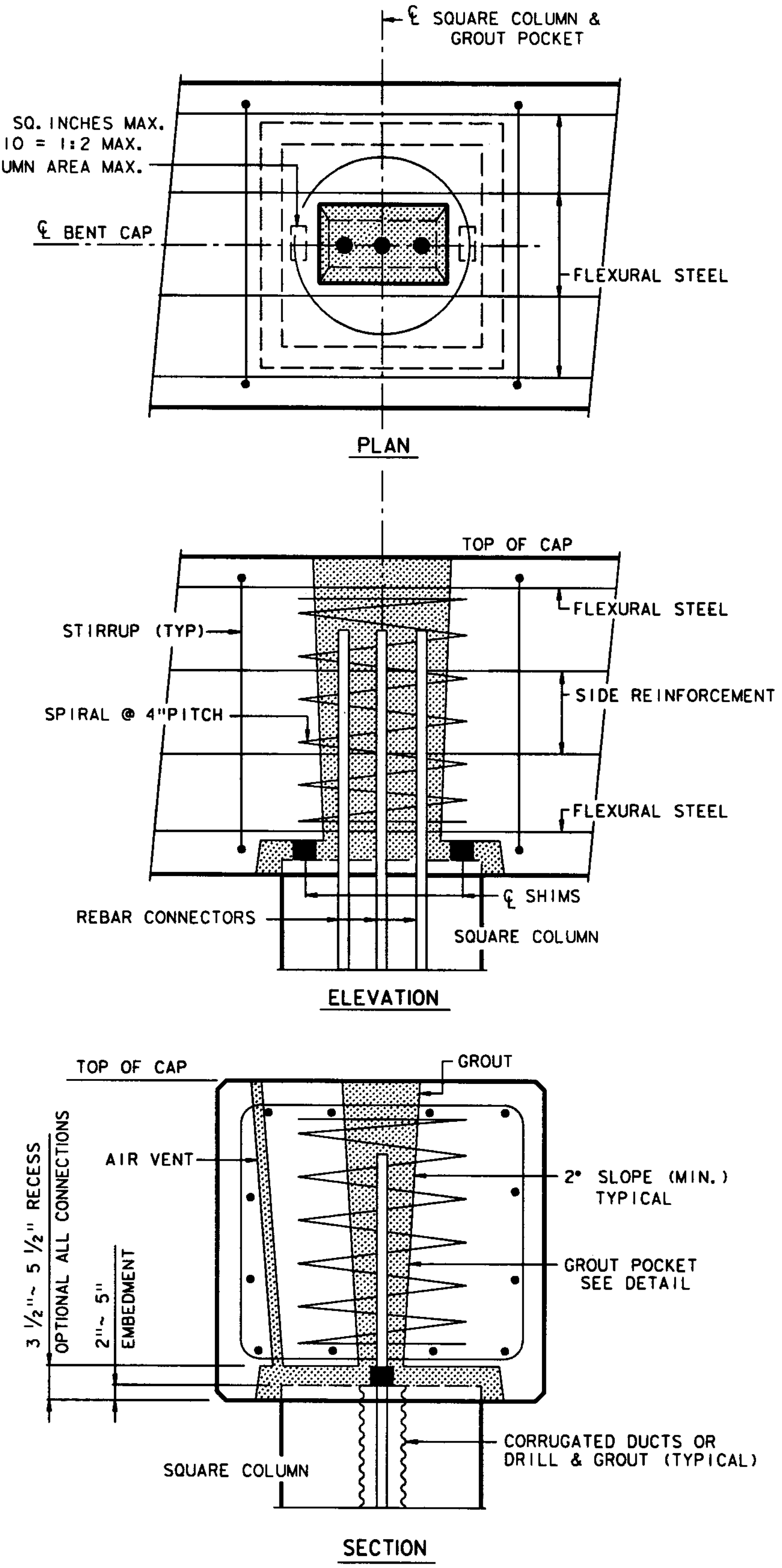


Figure 8.1 Single-line Grout Pocket on Pile (Embedded Option)

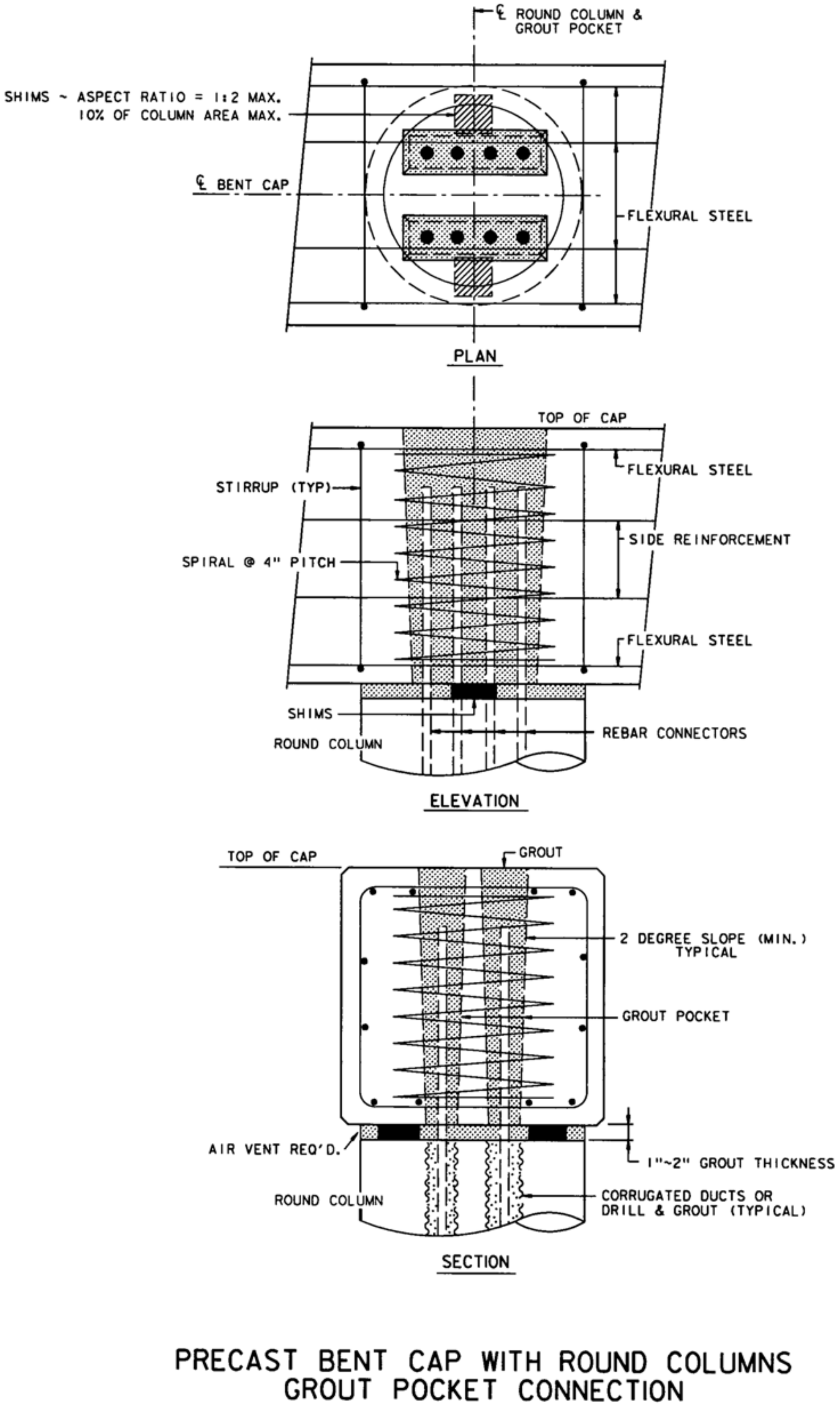


Figure 8.2 Double-line Grout Pocket on Column (Surface-flush Option)

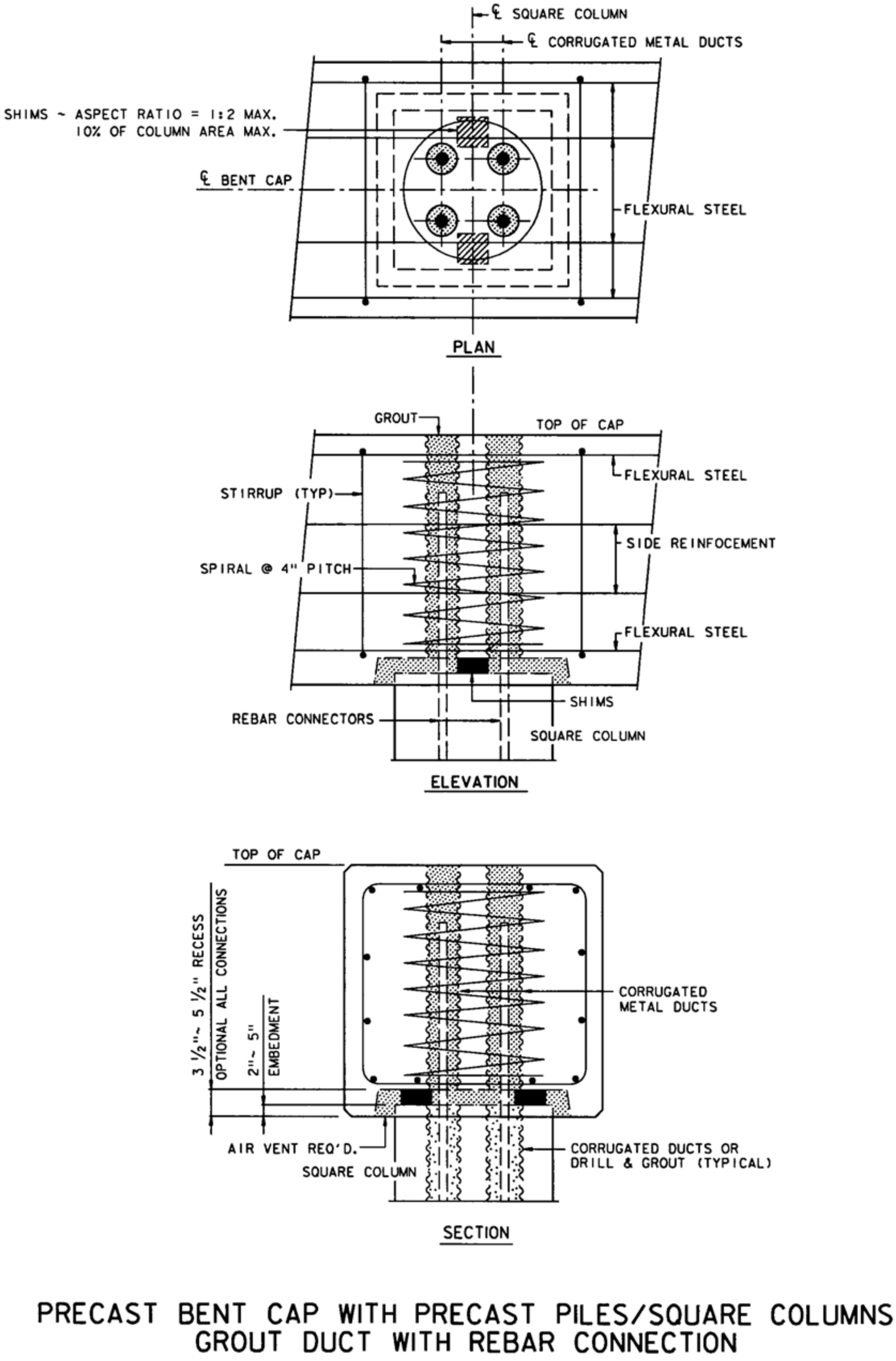


Figure 8.3 Grouted Vertical Duct on Pile (Embedded Option)

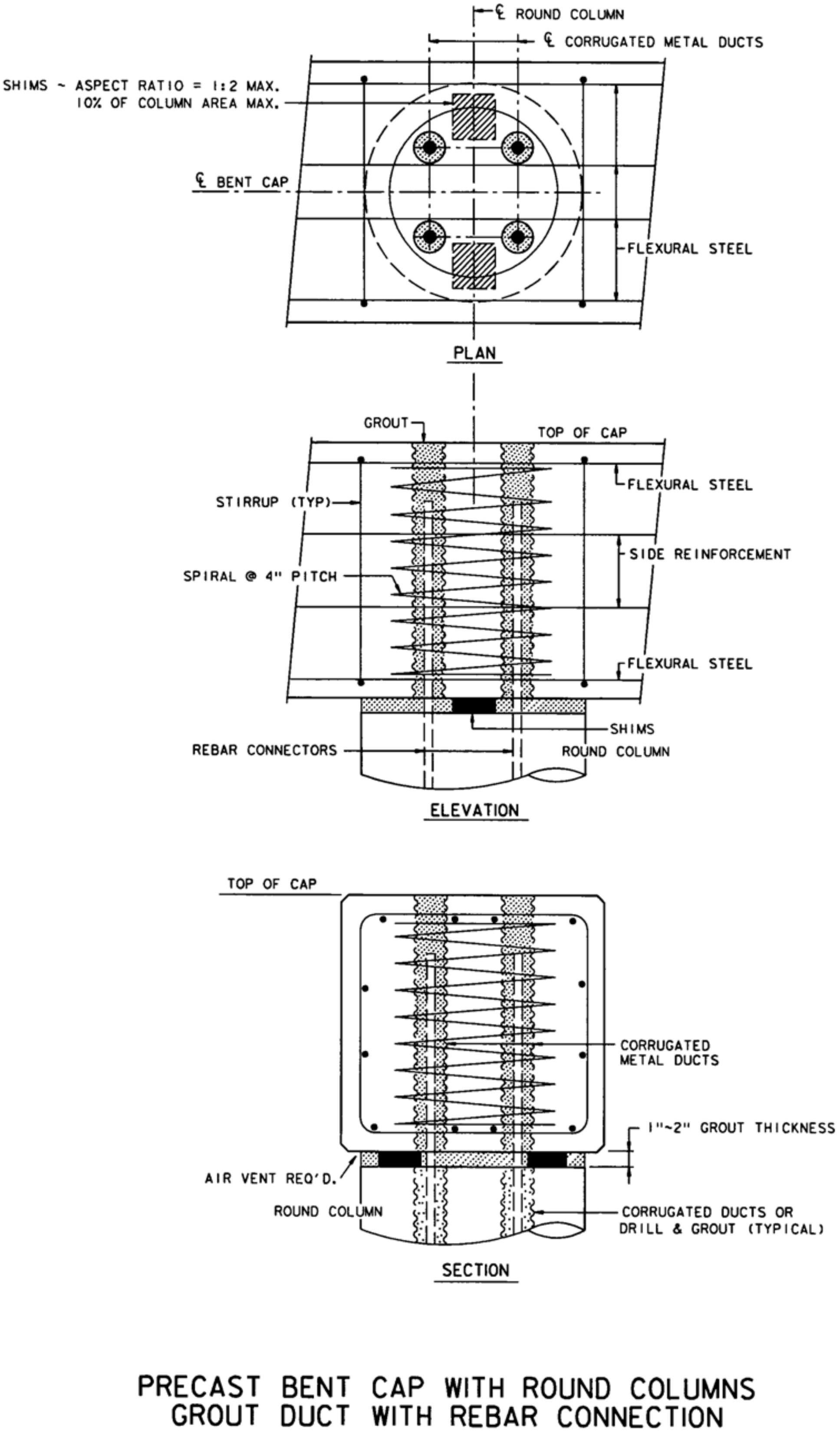


Figure 8.4 Grouted Vertical Duct on Column (Surface-flush Option)

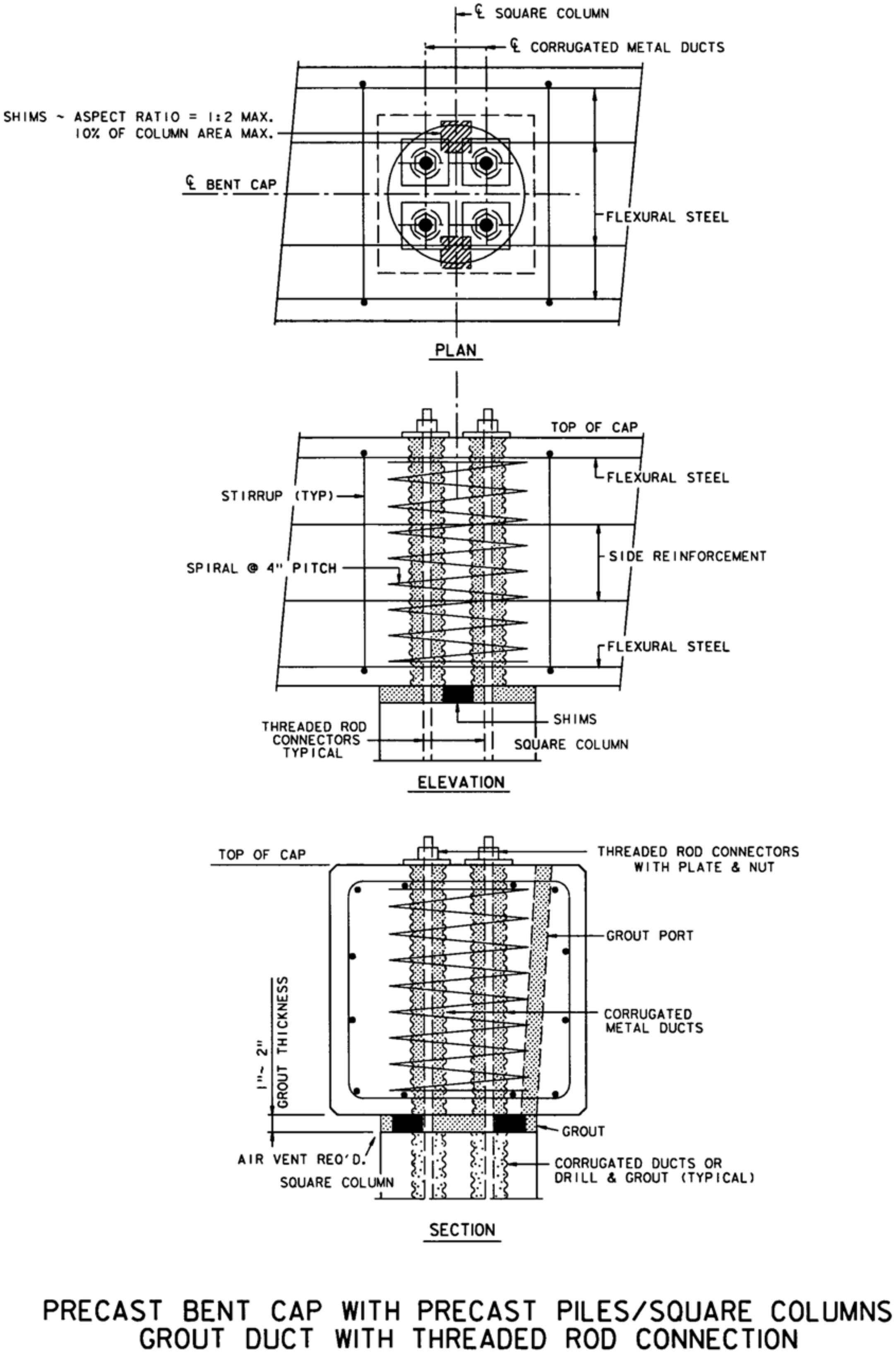


Figure 8.5 Bolted Connection on Pile (Surface-flush Option)

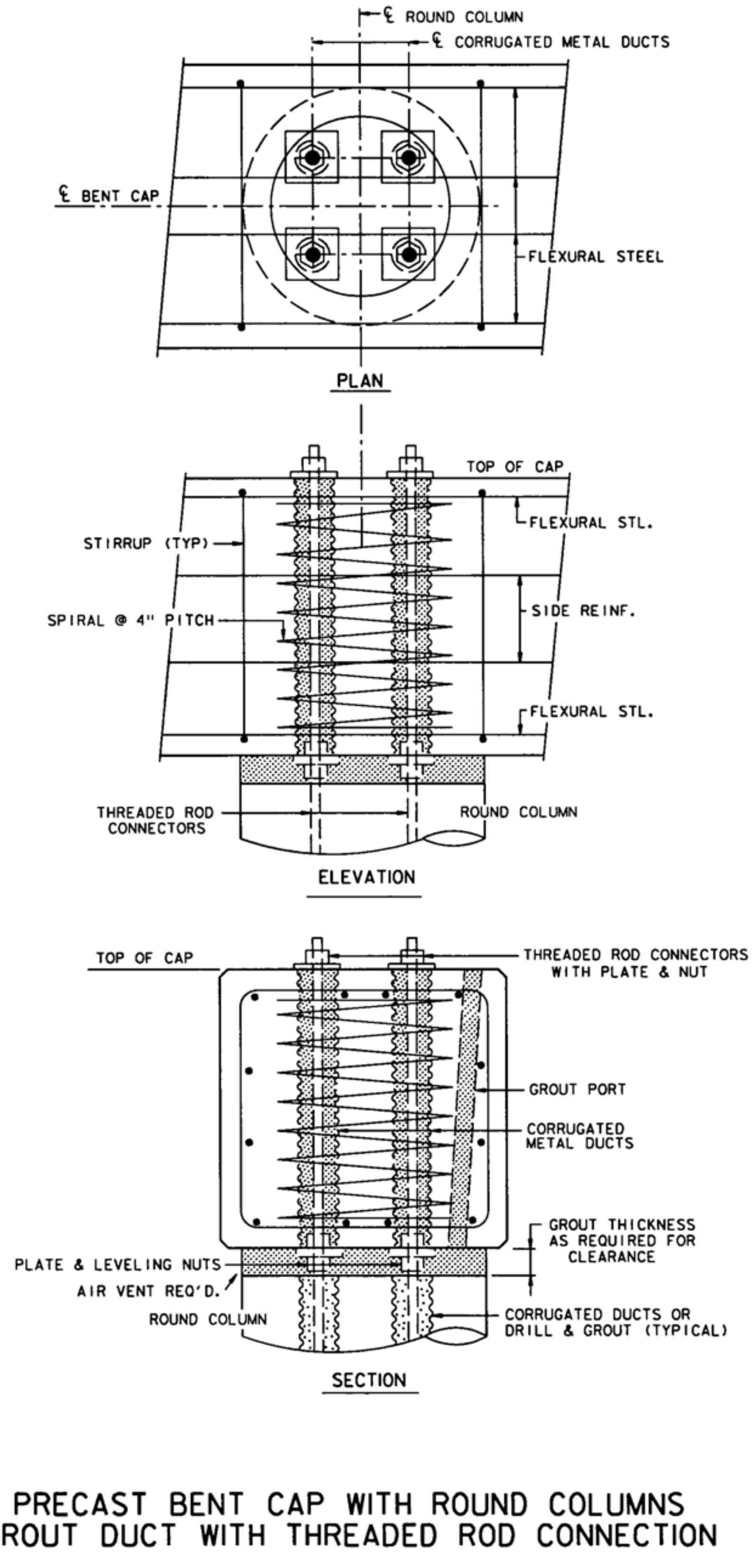


Figure 8.6 Bolted Connection on Column (Surface-flush Option; Plate-and-Leveling Nut Option)

8.5 REFERENCES

8.1. Texas Department of Transportation, *Standard Specifications for Construction of Highways, Streets, and Bridges*, Austin, TX, 1993.

CHAPTER 9:
SUMMARY AND CONCLUSIONS

9.1 SUMMARY

To help address concerns of construction time and safety related to traffic control and disrupted traffic flow, TxDOT has gradually introduced precast concrete components in bridge systems. Greater potential for economy and speed of construction, as well as benefits related to controlled conditions at precast plants, are anticipated through the development of a precast bent cap system. Two recent TxDOT projects that used precast bent caps convinced engineers and contractors that a precast bent cap system can significantly reduce construction time and cost, but concerns were expressed over design, detailing, behavior, constructability, and durability of the system.

Recognizing the potential benefits as well as uncertainties associated with a precast bent cap system, researchers from Ferguson Structural Engineering Laboratory at The University of Texas at Austin conducted research under TxDOT Project 0-1748 to accomplish the following objectives:

1. Develop practical, cost-effective candidate details for connecting precast bent caps to cast-in-place columns and precast trestle piles

2. Test select candidate connection details to examine connection constructability and behavior under loading

3. Develop a design methodology, example details, and construction recommendations for connecting precast bent caps to cast-in-place columns or precast trestle piles.

For practical reasons, the scope entailed only precast bent caps on precast trestle piles or cast-in-place columns in multi-column bents. Issues associated with seismic or other highly dynamic loads or bents of unusual proportions or applications were not addressed. In addition, focus was placed on systems requiring simple construction operations.

The research presented in this dissertation summarizes the accomplishment of these objectives, including:

- Review of previous TxDOT projects and relevant literature (Chapter 1)
- Development of candidate connection details in coordination with an Industry Review Committee consisting of representatives of the precast and construction industries as well as TxDOT engineers (Chapter 2)
- Experimental testing of candidate connection details in three phases to examine constructability and behavior:
  - Phase 1—thirty-two large-scale pullout tests examining the behavior and failure modes for grout pocket and grouted vertical duct connections (Chapter 3)
  - Phase 2—tests on four full-scale bent cap-to-column connections for grout pocket, grouted vertical duct and bolted connections (Chapter 4)
  - Phase 3—construction and testing of two full-scale bents using grout pocket, grouted vertical duct, and bolted connections at the construction yard of a highway contractor (Chapter 5)
  - Development of a design methodology for a precast bent cap system, based on test results from Phases 1-3 (Chapter 6)

- Development of a precast connection specification based on construction experience in Phases 1-3 (Chapter 7)
- Presentation of design recommendations, a precast connection specification, and example details for grout pocket, grouted vertical duct, and bolted connections (Chapter 8).

The following subsections summarize the work completed in the research program. Primary test results are included in the Conclusion section.

9.1.1 Review of Previous Projects and Relevant Literature

As a first step, two recent TxDOT projects that used precast bent caps, the Pierce Street Elevated project in Houston and the Red Fish Bay project on State Highway 361 near Port Aransas, were reviewed. Although there many aspects that led to the success of these projects, numerous challenges and uncertainties were identified. Literature related to precast substructures, precast connections, and anchorage of grouted and cast-in-place bars was also reviewed.

9.1.2 Development of Connection Details

Four categories of connections expected to be viable candidates for a precast bent cap system were identified. Categories included grout pockets, grouted vertical ducts, grouted sleeve couplers, and bolted connections. The Industry Review Committee provided input to increase the likelihood that connection details developed in the study would be constructible, durable, and economical. Advantages and disadvantages were identified, as well as uncertainties. Connector types, such as straight and headed bars and threaded rods, were identified. The essential components of a grout specification for prepackaged, non-shrink grout were established in relation to four properties: mechanical properties, compatibility, constructability, and durability. Three prepackaged grouts that appeared to satisfy this preliminary grout specification were identified and tested in trial batches using a pea-gravel extension. Mixes appeared to have potential for application to a precast bent cap system, although for constructability and liability reasons, neat grout (i.e., without extension) was preferred. Gravity-flow grouting using either a bucket or tremie tube was considered a simple, convenient approach, although concern was expressed regarding the difficulty of grouting connections without entrapping air.

9.1.3 Phase 1 Pullout Tests

Thirty-two pullout tests were conducted in Phase 1 on single-line and double-line grout pockets, as well as grouted vertical ducts to investigate uncertainties associated with connector anchorage. All tests used epoxy-coated bars and the majority were conducted using headed bars. Initial tests were conducted in pairs to compare response between neat and extended grouts for a single bar loaded in monotonic tension to failure, and to determine the depth at which failure transitioned from a concrete breakout failure to yield of the connector steel. Several comparisons were made to investigate the effect of straight vs. headed bars, two bars loaded simultaneously, confining effects of spiral reinforcement and welded wire fabric, and a cast-in-place specimen. Six double-line grout pocket pullout tests allowed different connector configurations to be investigated, although most of the variables were similar to single-line tests. Eight pullout tests were also conducted on grouted vertical duct specimens using straight and headed bars. Grout brand, grout strength, grouting operations, and interlock between grout pockets, ducts, and bars were examined.

9.1.4 Phase 2 Connection Tests

A series of tests were conducted on four full-scale precast bent cap-to-column connections using a single-line grout pocket, double-line grout pocket, grouted vertical duct, and bolted connection. Three specimens used a cast-in-place column with the cap, and the other used a pile. Both constructability and behavior under loads were investigated through fabrication of a precast cap and column stub, cap placement, connection grouting, and testing. Constructability issues included construction tolerances, shim stability during cap setting, gravity-flow grouting of multiple pockets, voids, and the bedding layer using bucket and tremie-tube approaches, and grout performance. Behavioral issues were related to connection behavior under realistic service-level and factored loads, connector embedment depth and anchorage, connection strength and ductility, confining reinforcement, bedding layer response, potential load concentration at shims, grout type and strength, and comparison to analytical predictions.

9.1.5 Phase 3 Bent Tests

Phase 3 included construction and testing of a trestle pile bent and a column bent at the construction yard of the contractor. Two bents incorporated a total of seven connections, including single-line and double-line grout pockets, grouted vertical ducts, and bolted connections. Construction issues were similar to Phase 2, but also included use of a grout specification, construction tolerances for multiple connections, suitability of a plate-and-leveling-nut option for cap setting, trial grout batches, grouting of pile and column ducts, grouting of multiple connections, grouting under adverse weather conditions, and contractor-engineer interaction. Behavioral issues were similar to Phase 2.

9.1.6 Development of a Design Methodology

A design methodology was developed for a precast bent cap system using the results from Phases 1-3. After describing the design philosophy, a design procedure was presented that included the following eight steps: 1) selection of a trial bent configuration, 2) analysis of bent cap and columns, 3) determination of connection actions, 4) selection of connection type and embedment, 5) selection of trial connector configuration, 6) analysis of connector configuration, 7) determination of connector type and embedment, and 8) determination of confining reinforcement and auxiliary reinforcement. In addition, design equations for anchorage of straight or headed connectors in grout pockets or grouted vertical ducts were also developed based on data from Phase 1 pullout tests.

9.1.7 Development of a Precast Connection Specification

Based on the lessons learned during construction of the specimens in Phases 1-3, a precast connection specification was developed. Rationale for major components of the specification was presented and included sections on materials, a precast bent cap placement plan, grouting operations, and other related items. Modifications were incorporated in the grout specification introduced in Chapter 2. The Precast Bent Cap Placement Plan addressed requirements for construction sequence, grouting operations, bent cap support, and manufacturer’s literature. Requirements for grouting operations addressed trial batches, formwork, presoaking, a pre-grouting meeting, and grouting methods.

9.1.8 Design Recommendations, Precast Connection Specification, and Example Connection Details

Based on the development and commentary provided in Chapters 6 and 7, the final products of the research were presented: design recommendations, a precast connection specification, and example connection details. Design recommendations are a consolidation of Chapter 6 design provisions in code format for direct application to design. The precast connection specification is written in the standard format of TxDOT construction specifications. In addition, example connection details that conform to the design recommendations and the precast connection specification are shown for grout pocket, grouted vertical duct, and bolted connections. The appendix provides plan sheets for the first bridge designed using the design recommendations.

9.2 CONCLUSIONS

Research conclusions are divided into two categories: construction and behavior. Design recommendations, construction guidelines, and example connection details are provided in Chapter 8.

9.2.1 Construction

9.2.1.1 Fabrication

1. Fabrication of precast bent caps with grout pockets and corrugated ducts is feasible and relatively simple. Removable wood forms for grout pockets are convenient for small, one-time applications. Mass production and reuse favor removable steel forms. Stay-in-place forms are also viable.
2. The diameter of ducts embedded in piles and columns should be maximized to provide sufficient tolerances for connector placement. Ducts should provide a tolerance of at least +/-0.75 in. for connectors, although a 1-in. minimum is preferable. Potential conflict between ducts and pile or column reinforcement must be considered.
3. Accurate positioning of ducts and connectors may be achieved using templates and/or supplementary reinforcement. Templates should include a central opening to facilitate casting and may be fabricated with the ducts in a one-piece assembly.
4. Compressibility of the spiral reinforcement (like a coil spring) facilitates placement of spiral confining reinforcement. Preparation and placement of welded wire fabric are more difficult and thus welded wire fabric is not recommended for use as confining reinforcement.

9.2.1.2 Cap-Setting Operations

1. Cap-setting operations for multiple connections presented no difficulties.
2. Shims, friction collars, and plates with leveling nuts are viable alternatives to assist in cap setting.
3. Steel shims can provide better stability than plastic shims. Two shims can be used at exterior columns to improve bearing reliability. Shims should be glued or otherwise attached to the surface to prevent shifting.
4. Ducts embedded in the cap and columns (or piles) should be sized to provide horizontal construction tolerances of at least +/-1 in. Tolerances should account for combined tolerances and clearances associated with placement of connectors in piles or columns, fabrication and placement of pockets and ducts in the bent cap, as well as grouting operations.
5. Guide pipes can be used to facilitate cap setting over multiple connectors.

9.2.1.3 Trial Batches

1. Trial batches of candidate grouts are essential to verify grout properties and compare grout brands, as well as to provide experience for contractor personnel. Trial batches should use proposed grouting procedures and equipment, and should be conducted sufficiently early to ensure lessons learned are implemented in actual grouting operations.
2. Trial batches should establish the required volume and temperature of water to achieve a flow (i.e., efflux) time of 20-30 seconds based on the CRD-C 611/ASTM C 939 Flow Cone Method at the expected on-site air temperature. A representative portion of the grout should be used in the flow cone test to accurately determine grout flow after mixing and at the working time (i.e., pot life) of the grout. Specified strength and strength gain should also be achieved. Grouts should be inspected for excessive segregation or bleeding.
3. Grout selection should ensure that the grout provides an adequate working time, based on time required to conduct the flow cone test, transfer grout from the mixer to dispensers, transport grout to point of placement, and grout one or more connections.

9.2.1.4 Grouting Operations

1. Grouting procedures should be listed step-by-step in plan sheets to ensure procedures are accurately implemented.
2. An on-site pre-grouting meeting between the contractor and engineer or other representative should be conducted just prior to actual grouting operations to review the grouting procedure and ensure lessons learned during the trial batch process are incorporated. The engineer or other representative should be available for consultation during initial grouting operations and periodically thereafter.
3. Grouting should be limited to the same weather restrictions as for casting concrete.
4. Prewatering connection voids is essential to ensure forms are tight, as well as to prevent moisture loss from grout.
5. Grouts should not be remixed and water should not be added to previously mixed grout. A 0.5-in. wire mesh should be used to ensure potential clumps are removed from grout after mixing. A flow time of 20-30 seconds should be achieved, based on the CRD-C 611/ASTM C 939 Flow Cone Method.
6. Gravity-flow grouting is feasible for all connection types. Grout pockets may be grouted using either a bucket or tremie tube. Connections with ducts should use tremie-tube grouting to prevent air entrapment. Grout pumping is also expected to be viable.
7. To ensure continuous flow during tremie-tube grouting, grout should be mixed in a sufficient quantity and poured using a funnel of sufficient volume.
8. Vent tubes located at the top of the bedding layer should be used to vent air.
9. Shims should be placed at least 2 in. from edges of columns or piles to facilitate grouting of the bedding layer. Total shim area should be limited to approximately 10% of the column or pile area.
10. Connections should be tamped but not vibrated during grouting.
11. Exposed grout surfaces should be carefully examined for voids after grout setting and form removal.
12. A 1.5-in. thick bedding layer is adequate for conducting grouting operations.

9.2.2 Behavior

Because design recommendations address the overall design approach, conclusions on behavior are limited to issues from Phase 1-3 tests. Italicized conclusions are followed by a brief synopsis of test-related behavior that supports the conclusion.

9.2.2.1 General Behavior

1. *Single-line and double-line grout pocket, grouted vertical duct, and bolted connections are all acceptable connection alternatives for a precast bent cap system.* For all connection types, the expected strength was achieved and excellent ductility was exhibited. The yield strength of bars was developed with limited slip. The effect of a bedding layer and shims on response was negligible. The influence of using a different number and configuration of connectors than for a cast-in-place bent can be conservatively accounted for in design.
2. *Minor levels of connector strain and cracking in the connection region are expected for service and factored loads at eccentricities as large as 6 in. in the transverse direction (relative to the*

centerline of the bridge superstructure) and 7.25 in. in the longitudinal direction. For larger eccentricities, connector yield may occur.

3. *Straight reinforcing bars can be designed for use in grout pocket connections in a precast bent cap system.* In grout pocket pullout tests, epoxy-coated straight bars exhibited either a pullout failure or steel yielding in grout pocket pullout tests. Confining effects of the surrounding concrete on the grout contributed to the achievement of bar yield within a depth of approximately $12d_b$ for concrete strengths of approximately 5.4 ksi. Pullout failures were accompanied by significant splitting cracks in the grout pocket and surrounding concrete as well as a shallow cone of grout and concrete. Capacity was established using a uniform bond stress model. Design recommendations conservatively require an embedment depth of $22d_b$ to develop the yield strength of connectors (60 ksi steel, 3600-psi concrete). Typical depths of bent caps can easily accommodate this requirement. Because epoxy-coated bars were used in tests, design provisions are conservative when applied to uncoated bars (this applies to other connector types as well).

4. *Headed reinforcing bars can be designed for use in grout pocket connections in a precast bent cap system.* In grout pocket pullout tests, epoxy-coated headed bars exhibited either a concrete breakout failure or steel yielding. Confining effects of the surrounding concrete on the grout contributed to the achievement of bar yield within a depth of approximately $12d_b$ for concrete strengths of approximately 5.4 ksi. Breakout failures exhibited a failure surface that emanated from the head of the bar but was accompanied by significant splitting cracks and grout pocket corner cracks in the grout pocket and surrounding concrete. Capacity for single or multiple bars was predicted closely by the Concrete Capacity Design Method, using a cracking factor of 0.75 to account for additional cracking in the grout pocket region. However, the embedment depth need not exceed that required for straight bars.

5. *Straight and headed reinforcing bars can be designed for use in grouted vertical duct connections in a precast bent cap system.* In grouted vertical duct pullout tests, straight and headed epoxy-coated bars exhibited either pullout failure or steel yielding. Confining effects of the surrounding concrete on the duct contributed to the achievement of bar yield within a depth of approximately $13d_b$ for concrete strengths of approximately 5.5 ksi. Pullout of the grout-bar unit from the duct was initiated after splitting cracks reduced confinement of the concrete surrounding the duct, leading to duct dilation. Capacity was established for straight bars using a uniform bond stress model. Design recommendations conservatively require an embedment depth of $24d_b$ to develop the yield strength of connectors (60 ksi steel, 3600-psi concrete). At an embedment depth sufficient for bar yield, headed bars exhibited little difference in response from straight bars. Therefore, although headed bars provide reserve anchorage capacity because of the head, design recommendations for embedment depth are the same as those used for straight bars.

6. *A ductile failure is expected for bent cap-to-column connections subjected to very large eccentricities. Strength can be accurately predicted from traditional approaches used for cast-in-place concrete. Anchorage of connectors can be conservatively predicted using pullout test results.* Bent-cap to column (or pile) connection tests required application of an unrealistically large eccentricity to produce connector yield. Failure loads were predicted within 15% for connection specimens and within 20% for a pile bent, based on axial load-moment (P-M) strength interaction diagrams. Substantial ductility was indicated in connection tests by the spread of bar yield from the bedding layer to a depth of at least 7.5 in. into the connection region of the bent cap as well as the columns or piles. The pile bent test also exhibited substantial ductility, as indicated by load-deflection response and flexural cracking in the region of the bedding layer and pile. Anchorage of epoxy-coated straight bars was sufficient to develop 1.25 times the bar yield strength at an embedment depth of $13d_b$ (concrete compressive strength of 6.0 to 7.2 ksi) and was maintained without evidence of splitting cracks in the region where the connectors were anchored.

9.2.2.2 Additional Conclusions from Pullout Tests

1. *Grout extension is not expected to produce a significant difference in capacity or behavior.* In grout pocket tests, nearly identical capacities were achieved for headed bars embedded in Masterflow 928 neat grout compared to Masterflow 928 grout extended with 3/8-in. pea gravel. Interlock between the grout pocket and concrete interface developed for both cases, without surface roughening. Coarse aggregate in the grout matrix slightly reduced the effects of splitting cracks on slip and enhanced stiffness, but neat grout demonstrated adequate overall stiffness and strength for use in connections.
2. *Significant differences in strength are expected between different grout brands and for grout batches using the same grout brand. However, these differences are not expected to produce a difference in anchorage behavior as long as the grout strength is sufficient to transfer load into the surrounding concrete.* A wide variation in grout strength was obtained for Masterflow 928, Sika 212, Euclid Hi-Flow neat grouts, as well as for different trial mixes of the same grout brand. In many specimens, the grout strength was less than that of the concrete, but did not adversely affect behavior. However, for one test (Phase 1, VD04) a grout strength of approximately 3 ksi did change the performance and failure mode. To ensure adequate transfer of load into the concrete, the precast connection specification requires that the modified grout cube strength exceed the concrete compressive strength by a 1000-psi margin. This same conclusion applies for connection and bent tests, which used lower grout strengths without adversely affecting connection performance.
3. *Negligible slip is expected between reinforcement bars and threaded rods, grout, steel corrugated ducts, and concrete.* Negligible slip was detected between bars, grout, ducts, and concrete during pullout tests. This same conclusion applies for connection and bent tests.
4. *For connectors embedded sufficiently for bar yield, stiffness and anchorage behavior for straight bars is expected to compare closely to that for headed bars.* Grout pocket and grouted vertical duct pullout tests indicated little difference between straight and headed bars when the embedment depth was sufficient for bar yield. This was due to the significant contribution of surface deformations of the bars (i.e., lugs) to bond. For embedment depths that will be used for a precast bent cap system, heads provide reserve strength in the event of a loss of bond along the bar length.
5. *Bars in a grout pocket are expected to achieve a slightly smaller capacity than cast-in-place bars.* Grouted bars achieved a capacity 20% less than that for cast-in-place bars in a grout pocket comparison test. In addition, grouted bars exhibited softer load-slip response and greater bond degradation, related to more extensive cracking in the grout pocket region.
6. *Use of spiral confining reinforcement around grout pockets can increase capacity and ductility.* A grout pocket pullout specimen with spiral confining reinforcement achieved a capacity approximately 50% greater than an unconfined specimen, and a displacement ductility approximately 250% greater. In addition, confinement limited crack growth and crack widths prior to failure. Welded wire fabric also produced greater capacity and, at significantly larger deformations than the spirally-reinforced specimen, greater ductility. As mentioned subsequently, this conclusion does not necessarily apply to all connector sizes and configurations in a bent cap connection. For many cases, confining reinforcement may not be mobilized. Welded wire fabric is not recommended as confining reinforcement for constructability reasons.
7. *Very shallow embedment depths for small bars in grout pockets can lead to a greater loss of bond due to splitting cracks.* The use of a bar size as small as a #6 in a grout pocket led to very shallow embedment depths of only 6 in. ($8d_b$) for some tests and more severe bond degradation upon the formation of splitting cracks in the grout. Design recommendations require a minimum

bar size of #7 and an embedment depth of at least 18 in. Thus, this effect is not expected for bar sizes and the depths that will be used in a precast bent cap system.

9.2.2.3 Additional Conclusions from Connection and Bent Tests

1. *The existence of a bedding layer up to 4 in. thick is not expected to adversely affect connection response. Potential cracks at the exposed surface of the bedding layer are not expected to pose structural or durability concerns.* Response in connection and bent tests did not indicate that the presence of a bedding layer affected strength, ductility, or stiffness at service or factored level. In connection tests, vertical cracks formed at the exposed surface of the bedding layer at service and factored levels during proof tests. However, crack widths were limited to 0.002 in. and are therefore not of concern from a structural or durability perspective. In pile and column bent tests, vertical cracks did not form on the exposed portion of the bedding layer under factored loads, even for the 4-in. bedding layer used in the bolted connection on the column bent.
2. *Connections that have the pile or column top embedded into the bottom of the bent cap are expected to exhibit response similar to connections that have the pile or column top (with the bedding layer) surface-flush with the bottom of the cap. The embedded connection alternative is expected to provide better durability protection.* In pile bent tests, similar response occurred for the embedded and surface-flush connection options. For large longitudinal eccentricities, similar response is attributed to rotation of cap. No discernable differences were evident in other connection or bent tests. Compared to a surface-flush connection, an embedded connection is expected to provide better durability protection because of the longer path for moisture ingress at the bedding layer and lack of opening or cracking at service level loads. Such connections, however, prevent inspection after grouting operations.
3. *Use of shims or plates and leveling nuts is not expected to adversely affect connection response or capacity.* The use of steel or plastic shims or plates and leveling nuts in the bedding layer did not produce any discernable effect on connection response in proof tests or failure tests for connection and bent specimens.
4. *Spiral confining reinforcement is not expected to be mobilized by typical connection design forces.* In connection and bent tests using grout pockets, a connector clear spacing as small as approximately $2d_b$ to $2.5d_b$ was used with an embedment depth of $13d_b$. Splitting cracks did not develop nor did connectors appear to slip even for failure tests. Connection and bent tests used clear spacing between ducts as small as one duct diameter, without formation of splitting cracks or connector slip in failure tests. With larger embedment depths used in design, splitting cracks are not expected. In addition, connectors are not expected to be challenged severely for realistic eccentricities. For closer connector or duct spacing, design provisions require a 50 percent increase in connector embedment depth. Thus, contributions of confining reinforcement to ductility and strength are expected to be minor. Minimum confining reinforcement, however, is still recommended for use in the connection region as a conservative, inexpensive provision to account for the paucity of test data and to provide confinement and crack control for unexpected events.
5. *Traditional approaches can be used to determine connection strength for combined flexure and axial force effects, including the effects of biaxial bending. Traditional reinforced concrete section models can be used to predict moment-curvature response for particular connector arrangements. However, prediction of load-deflection response depends on an accurate determination of connection stiffness.* Failure loads were predicted closely (within 10-20%) for connection and bent tests using axial load-moment (P-M) strength interaction diagrams and the Reciprocal Load Method. The predicted moment-curvature response also matched actual response reasonably well for connection tests. Deflections were not always reliably predicted in

tests. Insufficient data is available for accurate determination of connection stiffness for frame analysis. However, design actions and deflections can be conservatively determined.

9.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The scope of research for TxDOT Project 0-1748 was exceptionally broad, allowing only a select number of behavioral and construction issues to be examined. The following sections outline additional research that is needed to further develop a precast bent cap system.

9.3.1 System

1. Entirely precast substructure—A precast substructure, including columns, can provide an even greater reduction in construction time and should therefore be investigated. Research results provide a stepping stone for such a development.
2. Post-tensioning—A major advantage of using post-tensioning for a precast bent cap system is the use of several lighter bent cap segments, instead of a single, relatively heavy bent cap. This approach will enable longer bent caps to be used, especially for inverted-tee cross sections. In addition, the number and/or size of columns and foundations can be reduced, resulting also in improved aesthetics. Construction, behavior, design and detailing need to be addressed. Match-casting should be considered.
3. Bent cap voids—Large voids can be used in a bent cap to reduce weight. This would change the conventional design and detailing that the current system employs, but may not require major changes to connection design and detailing if the cap were solid in the vicinity of the column. This approach could also be applied to precast columns.
4. Structural lightweight concrete—Cap weight could also be reduced by use of structural lightweight concrete. Behavior, including anchorage capacity, would have to be verified.
5. Implementation—Clearly, the actual design and construction of a number of precast bent cap systems are necessary to assess the feasibility, ease, and speed of construction and to identify areas that need improvement, including the design methodology, precast connection specification, and connection details. The appendix provides the plan sheets for the first precast bent cap system designed by TxDOT using the provisions of this research. Based on a contractor request, a precast bent cap system was selected for several bents of the Lake Ray Hubbard Bridges on SH66 in the Dallas District bridge (rectangular bent caps on cast-in-place columns). Construction should be monitored. Implementation on a greater number and variety of projects is also needed.

9.3.2 Grouting

1. Grout pumping—Grout pumping through ports in the bedding layer, sidewall, or openings at the cap top should be investigated as an alternative grouting approach to the gravity-flow grouting methods investigated in the research. More reliable grouting of the bedding layer is expected, although a higher level of skill in conducting grouting operations may be required.
2. Other types of grout material—Other types of grout materials should be investigated, including prepackaged epoxy or polyester resin mixes. This would be especially valuable for cases in which better properties are achieved, such as longer working times and a suitable flow in hot or cold weather, lack of segregation, and enhanced durability. However, economic feasibility must be verified. To provide the contractor more options, additional brands of prepackaged grout should be identified for use. A non-proprietary grout mix could also be developed, but concerns over quality control would have to be addressed. Use of extended grouts or concrete with small aggregate should be used with caution. Appropriate schemes should be devised to ensure well-

defined properties, suitable workability, consolidation of the mix in the bedding layer, liability for the different components, and an economical product.

3. Durability properties of grouts—An investigation should be conducted to determine durability properties of interest for common prepackaged grout mixes, as little information is currently available for many grouts. Properties related to freeze thaw, sulfate resistance, permeability, and bleed should be established.
4. Sealants—Sealants such as epoxy or silene should be investigated for application to exposed portions of grouted connections.

9.3.3 Behavior

1. Anchorage—Pullout tests should be conducted to increase the data base for straight and headed bar anchorage. A small number of tests provide the basis for anchorage equations. Other variables that should be investigated include: uncoated bars, larger bar sizes, higher strength concrete, smaller duct corrugation height, additional grout brands, different grout pocket shapes, stay-in-place grout pocket forms, and different size and shape heads. Pullout tests should use larger specimen sizes, and additional cast-in-place comparison specimens should be tested.
2. Cast-in-place comparison tests—At least two cast-in-place comparison tests should be conducted to determine differences in response between a cast-in-place bent and a precast bent cap system. One specimen should use a connector configuration similar to that used in most connection tests (four #9 connectors in a 30-in. column). This is expected to verify minimal differences between cast-in-place and precast systems due to connector anchorage and the presence of a bedding layer, as well as provide a basis for connection stiffness. A second specimen should be based on standard practice of one percent longitudinal reinforcement with an embedment of the cap depth minus 6 in. This will also require a new precast bent cap specimen for comparison, also using one percent longitudinal reinforcement, but arranged in a way to investigate potential differences in strength, ductility, and stiffness.
3. Connectors and duct spacing—Connection tests should be conducted to determine the effect of closer spacing between connectors and ducts and to determine if confining reinforcement would increase ductility and strength for such cases. Tests were limited to connector spacing of $2d_b$ and duct spacing of one duct diameter.
4. Connection stiffness—Connection tests should be conducted to determine rotational stiffness of connections (moment-rotation response) for different connection types and connector configurations. Results should be used to help establish a relationship between experimental results and analytical predictions. This would provide designers more realistic results from analysis for connection and bent design.
5. Bedding layer—Connection tests should be conducted to determine the influence of different bedding layer thicknesses on connection strength and stiffness, including thicknesses greater than 4 in. Occasionally, a thicker bedding layer may be necessary in practice. The bedding layer did not affect strength or stiffness in tests for thicknesses as large as 4 in.
6. Loading—Effects of high-cycle fatigue loading should be investigated using specimens prepared for pullout tests and/or connection tests. Cyclic loading may also be investigated to account for highly-dynamic wind loading. A precast bent cap system for seismic regions should eventually be eventually developed. This will require many additional factors to be considered.

APPENDIX A

GROUT PRODUCT DATA SHEETS

1. **Euclid (Euco) High Flow**
2. **Masterflow 928**
3. **Sika 212**

EUCLID HI-FLOW GROUT

HIGH-TOLERANCE / NON-SHRINK GROUT

HI-FLOW GROUT is specially designed for use where high tolerance, high strength and high fluidity are required. It is formulated as a natural aggregate system with a shrinkage-compensating binder and is highly flowable without sacrificing strength or performance capabilities. HI-FLOW GROUT is formulated to provide consistent and exacting performance in critical grouting operations.

PRIMARY APPLICATIONS

- Heavy duty grouting of machinery and equipment
- Structural columns
- Crane rails
- Bridge seats
- Bearing plates
- Anchorages

FEATURES / BENEFITS

- Highly fluid and extremely placeable for easy field use
- High strength for maximum load bearing
- Non-shrink with minimum positive expansion for high-tolerance performance
- Non-bleeding and non-segregating at a fluid con- sistency
- Does not contain any chlorides or additives which may contribute to corrosion of base structure
- Total shrinkage compensation which provides a maximum bearing surface for the greatest overall support
- Rapid strength gain to minimize turnaround time for equipment regrouts
- Excellent working time at high ambient temperatures

PACKAGING / YIELD

HI-FLOW GROUT is packaged in 50 lb (22.7 kg) bags and yields 0.45 ft^3 (0.013m^3) of fluid grout when mixed with 1.2 gal (4.5 liter) of water.

TECHNICAL INFORMATION

Engineering Data

The following results were developed under laboratory conditions.
Tested at a fluid consistency, 1.2 gal of water/50 lb grout (4.7 liter/22.7 kg).
Compressive Strength ASTM C-109, 2"(50 mm) cubes

1 day ....4,000 psi (27 MPa)

3 days ....6,000 psi (40 MPa)

7 days ....7,000 psi (47 MPa)

28 days ....9,000 psi (61 MPa)

Volume Change ASTM C-1090 & CRD-C-621

1 day ....+.07%

3 days ....+.07%

7 days ....+.07%

28 days ....+.07%

Flow Rate ASTM C-939 & CRD-C-611
(defined as fluid by CRD-C-621 & ASTM C-1090)

Initial ....21 seconds

30 minutes ....29 seconds

60 minutes ....31 seconds

Setting Time ASTM C-191

Initial set ....3 hours, 50 minutes

Final set ......4 hours, 50 minutes

Flexural Strength ASTM C-78

3 days ....1,000 psi (6.8 MPa)

7 days ....1,200 psi (8.0 MPa)

28 days ....1,300 psi (8.8 MPa)

Split Tensile Strength ASTM C-496

28 days ....550 psi (3.7 MPa)

Stress Strain Analysis:

Tested in accordance with ASTM C-469 using 4" X 8" (100 mm at 200 mm) cylindrical specimens.

28 day ....see figure 1

Young's Modulus 4.1x106 psi (2.8x104 MPa)
Toughness Index vs. Plain Concrete

`at fc= 5,000 psi (35 MPa) ......2.4`

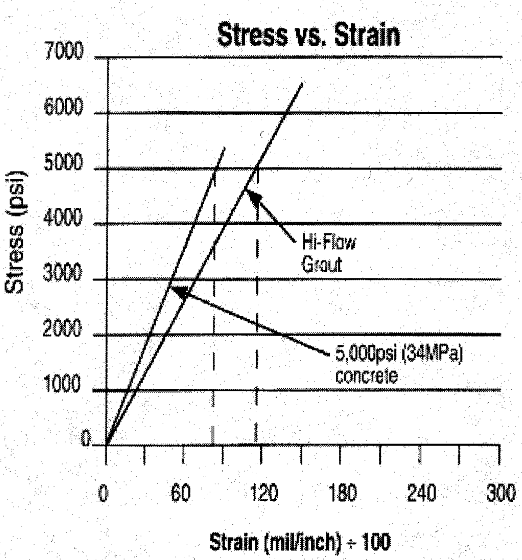


Figure 1

Appearance–
HI-FLOW GROUT is a free flowing powder designed to be mixed with water. After mixing and placing, the color may initially appear much darker than the surrounding concrete. While this color will lighten up substantially as the concrete cures and dries out, the grout may always appear somewhat darker than the surrounding concrete.

SPECIFICATIONS / COMPLIANCES

- Meets the requirements of CRD-C-621, Corps of Engineers Specification for Non-Shrink Grout.
- Shows positive expansion when tested in accordance with ASTM Specification C-1090, Standard Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout.
- Meets the performance requirements of ASTM C-1107, Grades A & B as well as Grade C, combination volume adjusting grout standard specification for packaged, dry, hydraulic-cement grout (non-shrinkable).

DIRECTIONS FOR USE

The contractor and engineer are encouraged to consult and review the Euclid Chemical bulletin "Application Instructions-Cementitious Grouting". The document offers instructions detailing the general installation of Euclid Chemical manufactured cement-based grout products.

Note: If the contractor is not familiar with standard grout placement techniques, a pre-job meeting is suggested to review the project details unique to the particular job. Contact your local Euclid Chemical Company representative for additional information.

The information given here is offered in particular support to the mixing and placing of HI-FLOW GROUT. This information should be used in conjunction with the Application Instructions guide mentioned above.
General Information-While HI-FLOW GROUT is designed to be fluid poured at temperatures ranges from 40-100°F (4.5-37.5°C) the product is most easily poured at temperatures of 60-70°F (16-21°C).

Mixing-Do not use this product at a flow cone rate of less than 20 seconds if checking flow rates on the job site (see CRD-C-611 or ASTM C-939 for flow cone method). Where HI-FLOW GROUT will be placed at a thickness over 4" (101.6 mm), up to 20 lb (9.1 kg) of pea gravel may be added to each bag of grout. Note that the water demand to achieve a certain flow level of the grout will change. Do not add sufficient water to promote bleeding of the grout.

Mixing Water Guide gal (liter)/bag

Consistency	Estimated Water Content*
Fluid	1.15-1.25 gal (4.35-4.73 liter)
Flowable	1.0-1.15 gal (3.79-4.35 liter)
Plastic	0.9-1.0 gal (3.41-3.79 liter)

*Do not add water in an amount that will cause bleeding or segregation. More or less water may be required to achieve a 25 second flow or the desired placing consistency, depending on temperature and other variables. Do not add sand or cement to the grout since this action will change its precision grouting characteristics.
Placing-HI-FLOW GROUT should be placed continuously.

Curing & Sealing-Proper curing procedures are important to ensure the durability and quality of the grout. Wet cure the grout until the forms are stripped. Then, cure the grout with a high solids curing compound, such as SUPER REZ-SEAL, SUPER FLOOR COAT or SUPER AQUA-CURE VOX as described in the general grouting Application Instruction guide.

CLEAN-UP

Clean tools and equipment with water before the material hardens.
Shelf life is 2 years in original, unopened package.

PRECAUTIONS / LIMITATIONS

- Store materials in a dry place.
- Proper curing is required.
- Do not add admixtures or fluidifiers.
- Do not use material at temperatures that may cause premature freezing.

- Keep the grout from freezing until a minimum strength of 4000 psi (28 MPa) is reached.
- Do not use as a topping.
- Employ cold weather or hot weather grouting practices as the temperature dictates.
- Shoulder cracking may occur on wide shoulders, improperly cured shoulders, or at stress points such as shimpacks, bolts or plate stiffeners. These cracks are of no structural significance.
- Rate of strength gain is significantly affected at temperature extremes.

Form-Hi-Flow Grout-12.97

The Euclid Chemical Company • Cleveland, Ohio 44110

FOR TRANSPORTATION & SAFETY EMERGENCIES CALL: 1-800-255-3924
INTERNATIONAL USERS CALL COLLECT: 1-813-979-0626

TRADE NAME
Hi-Flow Grout

CHEMICAL NAME
Portland Cement

MATERIAL	CAS#	%	ACGIH(TLV)	PEL
Silica Sand	**14808-60-7**	**45-50**	**0.1 mg/m3** **Respirable Dust**	**10 mg/m^3**
Portland Cement	**65997-15-1**	**30-35**	**10mg/m^3** **Total Dust**	
Alumino Silicate	**NE**	**10-15**		
Amorphous Silica	**7631-86-9**	**0-5**		

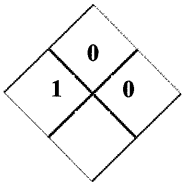
APPEARANCE **Gray Powder**	ODOR **None**	MELTPOINT **NA**	SPECIFIC GRAVITY **2.90-4.00**
VAPOR DENSITY **NA**	%VOLATILE BY WEIGHT **None**	BULK DENSITY **NA**	BOILING POINT **NA**
VAPOR PRESSURE **None**	%SOLUBILITY(H_2O)	EVAPORATION RATE	PH

FLASHPOINT & METHOD
NA

FLAMMABLE LIMITS
LEL **NA**
UEL **NA**

EXTINGUISHING MEDIA
NA

SPECIAL FIRE FIGHTING PROCEDURES
NA



UNUSUAL FIRE AND EXPLOSION HAZARDS
NA

Protection and Repair

Cementitious Grouts

MASTERFLOW® 928

Extended work-time, high-precision, non-shrink, natural aggregate grout

Description

MASTERFLOW® 928 grout is a high-precision, non-shrink, natural aggregate grout with extended working time. It is ideally suited for grouting machines or plates requiring precision load-bearing support. This specially formulated precision grout can be placed at a variety of consistencies - from fluid to damp-pack - over a wide temperature range - 45 to 90°F (7 to 32°C). MASTERFLOW 928 grout meets the requirements of ASTM C 1107 and the Army Corp of Engineers' CRD C 621, Grades B and C.

Recommended for

- Precision nonshrink grouting of:
  - Machinery and equipment, baseplates and soleplates
  - Precast wall panels, beams and columns, curtain walls, concrete systems, and other structural and nonstructural building members
  - Machinery, and concrete surface and crack repair in federally inspected meat and poultry plants
- Grouting anchor bolts, reinforcing bars and dowel rods
- Repairing concrete, including grouting voids, rock pockets and large cracks
- Applications requiring high one-day and later-age compressive strengths
- Applications requiring non-shrink grout to achieve maximum bearing for optimum load transfer
- Applications requiring grout to be pumped
- Marine applications
- Freeze/thaw environments

Features/Benefits

- Meets ASTM C 1107 and CRD C 621, Grades B and C, requirements at a fluid consistency over a temperature range of 45 to 90°F (7 to 32°C) over a 30 minute working time
- Can be mixed at a wide range of consistencies to ensure proper placement under a variety of application conditions
- Extended working time to ensure sufficient time for placement
- Hardens free of bleeding, segregation or settlement shrinkage
- Contains high quality, well-graded quartz aggregate for optimum strength and workability
- Sulfate resistant
- Freeze/thaw resistant

Packaging/Estimating

MASTERFLOW 928 grout is packaged in 55 lb (25 kg) moisture-resistant bags. It is also available in 3,300 lb (1,500 kg) bulk bags.

One 55 lb (25 kg) bag of MASTERFLOW 928 grout mixed with approximately 10.5 lb (4.8 kg) or 1.26 U.S. gal (4.8 L) of water yields approximately 0.50 ft^3 (0.014 m^3) of grout.

Note:The water requirement may vary due to mixing efficiency and other variables.

Performance Data

The following data was developed under controlled laboratory conditions. Reasonable variations from these results can be expected.

Typical Compressive Strengths (ASTM C 109, ASTM C 942, Modified)

	Consistency					
	Plastic[1]		Flowable[2]		Fluid[3]	
	psi	MPa	psi	MPa	psi	MPa
1 day	4,500	31	4,000	28	3,500	24
3 days	6,000	41	5,000	34	4,500	31
14 days	7,500	52	6,700	46	6,500	45
28 days	9,000	62	8,000	55	7,500	52

Volume Change (ASTM C 1090)

	% Change	Requirement per ASTM C 1107, %
1 day	0.04	0.0 - 0.30
3 days	0.04	0.0 - 0.30
14 days	0.05	0.0 - 0.30
28 days	0.06	0.0 - 0.30

Setting Time (ASTM C 191)

	Consistency		
	Plastic[1]	Flowable[2]	Fluid[3]
Initial Set (Hr.:Min.)	2:30	3:00	4:30
Final Set (Hr.:Min.)	4:00	5:00	6:00

Flexural Strength (ASTM C 78)*

	psi	MPa
3 days	1,000	6.9
7 days	1,050	7.2
28 days	1,150	7.9

Modulus of Elasticity (ASTM C 469, Modified)*

	psi	MPa
3 days	2.82×10^6	1.94×10^4
7 days	3.02×10^6	2.08×10^4
28 days	3.24×10^6	2.23×10^4

Coefficient of Thermal Expansion (ASTM C 531)*

6.5×10^{-6} in./in./°F (11.7×10^{-6} mm/mm/°C)

Split Tensile and Tensile Strength (ASTM C 496 and ASTM C 190)*

	Split Tensile		Tensile	
	psi	MPa	psi	MPa
3 days	575	4.0	490	3.4
7 days	630	4.3	500	3.4
28 days	675	4.7	500	3.4

[1] 100-125% flow on flow table per ASTM C 230

[2] 125-145% flow on flow table per ASTM C 230

[3] 25 to 30 seconds through flow cone per ASTM C 939

*Test conducted at a fluid consistency

Typical Ultimate Tensile and Shear Loads Anchor Bolt Tests (ASTM E 488)*

	Ultimate Tensile Load		Ultimate Shear Load	
	lb	kg	lb	kg
1-1/4 in. Bolts -	56,000	25,450	27,500	12,500
9 in. embedment in a 2-1/2 in diameter hole				
7/8 in. Bolts -	27,500	12,500	12,700	5,770
6 in. embedment in a 1-3/4 in diameter hole				
1/2 in. Bolts -	7,950	3,610	2,100	950
4 in. embedment in a 1-1/8 in diameter hole				

(Data based on threaded anchor bolts with washer and nut.
1 in. = 25.4 mm)

Punching Shear Strength (ChemRex Inc. Method)*

3 in. x 3 in. x 11 in (76 mm x 76 mm x 279 mm) beam

	psi	MPa
3 days	2,200	15.2
7 days	2,260	15.6
28 days	2,650	18.3

Resistance to Rapid Freezing and Thawing (ASTM C 666, Procedure A)

300 Cycles RDF 99%

*Test conducted at a fluid consistency

Installation

Consult the MASTERFLOW 928 grout Installation Bulletin and the product bag for details on the installation of MASTERFLOW 928 grout. ChemRex Inc. recommends that the user request the services of the local representative for a prejob conference to plan the installation.

Mixing

MASTERFLOW 928 grout should be mixed with a mechanical mixer for a least 5 minutes. For a fluid consistency, start with 9 lb (4 kg) [1.1 U.S. gal (4.2 L)] per 55 lb bag. Adjust mixing water, as needed, to establish the recommended flow of 25 to 30 seconds through a flow cone (ASTM C 939/CRD C 611). Less mixing water will be required to achieve stiffer consistencies.

Placing

MASTERFLOW 928 grout should be placed in a continuous pour. Discard grout that becomes unworkable. Grout should be placed from one side to avoid entrapment of air. Make sure that the grout fills the entire space to be grouted and remains in contact with the plate throughout the grouting process. Straps may be used to move the grout to ensure that the entire space is filled. DO NOT VIBRATE.

Curing

Immediately after placement, wet cure the MASTERFLOW 928 grout by covering all exposed grout with clean, damp rags (not burlap). Keep moist until grout surface is ready to be finished or until final set. Following the removal of the damp rags, immediately coat with a recommended curing compound, such as MASTERKURE® curing compound.

Jobsite Testing

If strength tests must be made at the jobsite, use 2 in. (51 mm) CUBE molds per ASTM C 109. DO NOT use cylinder molds. Testing should be controlled on the basis of the desired placing consistency rather than strictly on the water content. Consult with your local ChemRex Inc. representative for special procedures required when mixing and casting compressive strength tests of fluid, nonshrink grout.

Limitations

- The ambient and initial material temperature of the grout should be in the range of 45 to 90°F (7 to 32°C) for both mixing and placing. Ideally, the amount of mixing water used should be that which is necessary to achieve a 25 to 30 second flow per ASTM C 939 (CRD C 611). For placement outside of 45 to 90 °F (7 to 32°C), contact your local ChemRex Inc. representative.
- For pours greater than 6 in. (152 mm) deep, consult your local ChemRex Inc. representative for special precautions and installation procedures.
- For precision applications, requiring a fluid consistency (25 to 30 second flow per ASTM C 939/CRD C 611) with extended working time over a temperature range of 45 to 90°F (7 to 32°C) and dynamic load bearing support, use EMBECO® 885 grout.
- When the grout will be in contact with steel which is or will be stressed over 80,000 psi (550 MPa) use MASTERFLOW® 816 cable grout.
- MASTERFLOW 928, one-component, cement-based grout is formulated for industrial and professional use only, and must be kept out of the reach of children. This product contains chemicals which may be potentially HARMFUL to your health, if not stored and used properly. Hazards can be significantly reduced by observing all precautions which are found on Material Safety Data Sheets (MSDS), product labels, and technical literature. Please read this literature carefully before using product.

Customer Service: 1/800/433-9517
Technical Services: 1/800/ChemRex (1/800/243-6739)
Web Site: www.chemrex.com

Limited Warranty Notice

Every reasonable effort is made to apply ChemRex Inc. exacting standards both in the manufacture of our products and in the information which we issue concerning these products and their use. We warrant our products to be of good quality and will replace or, at our election, refund the purchase price of any products proved defective. Satisfactory results depend not only upon quality products, but also upon many factors beyond our control. Therefore, except for such replacement or refund, CHEMREX INC. MAKES NO WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF FITNESS FOR A PARTICULAR PURPOSE OR MERCHANTABILITY, RESPECTING ITS PRODUCTS, and CHEMREX INC. shall have no other liability with respect thereto. Any claim regarding product defect must be received in writing within one (1) year from the date of shipment. No claim will be considered without such written notice or after the specified time interval. User shall determine the suitability of the products for the intended use and assume all risks and liability in connection therewith. Any authorized change in the printed recommendations concerning the use of our products must bear the signature of the ChemRex Inc. Technical Manager.

Protection and Repair

ChemRex Inc.
889 Valley Park Drive; Shakopee, MN 55379

Manufacturing Plants: Newark, CA; Denver, CO; Fort Wayne, IN; Centerville, IN; Mattawan, MI; Bloomington, MN; Bristol, PA.

Regional Warehouses: Hayward, CA; Ontario, CA; Atlanta, GA; Chicago Heights, IL; Fairfield, NJ; Grand Prairie, TX; Brampton, ONT (Canada).

Form No. MB-169

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Not for sale to or use by the general public.

Printed on recycled paper including 10% post-consumer fiber

8M 3/00

MATERIAL SAFETY DATA SHEET

ChemRex, Inc.
Commercial Construction Products Division
889 Valley Park Drive
Shakopee, MN 55379

24-Hr Emergency
CHEMTREC (800) 424-9300

Prepared by: Regulatory Affairs Department (612) 496-6000
Revision Date: 03/29/00

Page: 1 of 4
Reason for revision: Manufacturer name change

This document is prepared pursuant to the OSHA Hazard Communication Standard (29 CFR 1910.1200). Where a proprietary ingredient is shown, the identity may be made available as provided in this standard.

All components of this product are included in the EPA Toxic Substances Control Act (TSCA) Chemical Substance Inventory.

1. PRODUCT NAME: MASTERFLOW 928 GROUT

Chemical Family: Hydraulic cement grouts..

2. HAZARDOUS INGREDIENTS:

		EXPOSURE LIMITS*			
	CASNO	TLV	STEL	PEL	CONTENT
Silica, Crystalline Quartz **	14808-60-7	***	None	****	30-60%
Portland Cement	65997-15-1	10 mg/M3	None	None	30-60%
Calcium Oxide	1305-78-8	2 mg/M3	None	5 mg/m3	< 5%
Silica, Amorphous	7631-86-9	3 mg/M3*****	None	None	< 5%

*) Refer to Section 7 for available LD/LC(50) Health Hazard Data.
**) Contains less than 0.1% w/w 53 micron or smaller Crystalline Quartz.
(***) 0.1 mg/m3 respirable quartz
(****) 10 mg/m3 divided by %SiO2+2 (respirable quartz)
(*****) Particulates NOC - Respirable

3. PHYSICAL DATA:

Boiling Point (oC):	N/Ap	Water/Oil Distribution Coefficient:	N/Av
Percent Volatile:	0		
Freezing Point (oC):	N/Ap	Solubility in Water:	Slight
Vapor Pressure mmHg @20(oC):	N/Av	Specific Gravity:	N/Av
Vapor Density:	> Air	pH:	N/Ap
Odor Threshold:	N/Av	Evaporation Rate:	N/Av

Appearance: Grayish granular powder
Odor: Odorless

N/Av = Not Available N/Ap = Not Applicable ca. = Approximate

4. FIRE AND EXPLOSION HAZARD DATA: HMIS Hazard Rating No. 0 (Minimal)

Flash Point: Non-flammable

Method: Not Applicable

Auto-Ignition Temp.: Not Applicable

ISO 9000

SikaGrout® 212

1/97

High performance, cementitious grout

DESCRIPTION

SikaGrout 212 is a non-shrink, cementitious grout with a unique 2-stage shrinkage compensating mechanism. It is non-metallic and contains no chloride. With a special blend of shrinkage-reducing and plasticizing/water-reducing agents, SikaGrout 212 compensates for shrinkage in both the plastic and hardened states. A structural grout, SikaGrout 212 provides the advantage of multiple fluidity with a single component. SikaGrout 212 meets Corps of Engineers' Specification CRD C-621 and ASTM C-1107.

WHERE TO USE

- ▲ Use for structural grouting of column base plates, machine base plates, anchor rods, bearing plates, etc.
- ▲ Use on grade, above and below grade, indoors and out.
- ▲ Multiple fluidity allows ease of placement: ram in place as a dry pack, trowel-apply as a medium flow, pour or pump as high flow.

ADVANTAGES

- ▲ Easy to use...just add water.
- ▲ Multiple fluidity with one material.
- ▲ Non-metallic, will not stain or rust.
- ▲ Low bleed.
- ▲ Low heat build-up.
- ▲ Excellent for pumping:
  1. Does not segregate...even at high flow.
  2. No build-up on equipment hopper.
- ▲ Non-corrosive, does not contain chlorides.
- ▲ Superior freeze/thaw resistance.
- ▲ Resistant to oil and water.
- ▲ Meets CRD C-621.
- ▲ Meets ASTM C-1107.
- ▲ Shows positive expansion when tested in accordance with ASTM C-827.
- ▲ SikaGrout 212 is USDA-approved.

COVERAGE

Approximately 0.48 cu. ft./bag at high flow.

PACKAGING

55-lb. multi-wall bags; 36 bags/pallet.

HOW TO USE

SURFACE PREPARATION

Remove all dirt, oil, grease, and other bond-inhibiting materials by mechanical means. Anchor bolts to be grouted must be degreased with suitable solvent. Concrete must be sound and roughened to promote mechanical adhesion. Prior to pouring, surface should be brought to a saturated surface-dry condition.

TYPICAL DATA FOR SIKAGROUT 212
(Material and curing conditions @ 73F and 50°₀ R.H.)

SHELF LIFE	One year in original, unopened bags.		
STORAGE CONDITIONS	Store dry at 40-95F (4-35C). **Condition material to 65-75F befor using.**		
COLOR	Concrete gray.		
FLOW CONDITIONS	**Plastic**[1]	**Flowable**[1]	**Fluid**[2]
Typical Water Requirements:	6 1/2 pt+	7 pt	9 1/2 pt
Set Time (ASTM C-266):			
Initial	3.5-4.5 hr	4.0-5.0 hr	4.5-6.5 hr
Final	4.5-5.5 hr	5.5-6.5 hr	6.0-8.0 hr
TENSILE SPLITTING STRENGTH, PSI (ASTM C-496)			
28 day	600 (4.1 MPa)	575 (3.9 MPa)	500 (3.4 MPa)
FLEXURAL STRENGTH, PSI (ASTM C-293)			
28 day	1,400 (9.6 MPa)	1,200 (8.2 MPa)	1,000 (6.8 MPa)
BOND STRENGTH, PSI (ASTM C-882 MODIFIED): Hardened concrete to plastic grou			
28 day	2,000 (13.7 MPa)	1,900 (13.1 MPa)	1,900 (13.1 MPa)
EXPANSION % (CRD C-621)			
28 day	+0.021%	+0.056%	+0.027%
COMPRESSIVE STRENGTH, PSI (CRD C-621)			
1 day	4,500 (31 MPa)	3,500 (24.1 MPa)	2,700 (18.6 MPa)
7 day	6,100 (42 MPa)	5,700 (39.3 MPa)	5,500 (37.9 MPa)
28 day	7,500 (51.7 MPa)	6,200 (42.7 MPa)	5,800 (40 MPa)

[1] ASTM C-230: 100-124% (plastic), 124-145% (flowable)

[2] CRD C-79: 10-30 sec efflux time.

FORMING

For pourable grout, construct forms to retain grout without leakage. Forms should be lined or coated with bond-breaker for easy removal. Forms should be sufficiently high to accommodate head of grout. Where grout-tight form is difficult to achieve, use SikaGrout 212 in dry pack consistency.

MIXING

Mix manually or mechanically. Mechanically mix with low-speed drill (400-600 rpm) and Sika mixing paddle or in appropriately sized mortar mixer.

MIXING PROCEDURE

Make sure all forming, mixing, placing, and clean-up materials are on hand. Add appropriate quantity of clean water to achieve desired flow. Add bag of powder to mixing vessel. Mix to a uniform consistency, minimum of 2 minutes. Ambient and material temperature should be as close as possible to 70F. If higher, use cold water; if colder, use warm water.

APPLICATION

Within 15 minutes after mixing, place grout into forms in normal manner to avoid air entrapment. Vibrate, pump, or ram grout as necessary to achieve flow or compaction. SikaGrout 212 must be confined in either the horizontal or vertical direction leaving minimum exposed surface. After grout has achieved final set, remove forms, trim or shape exposed grout shoulders to designed profile. SikaGrout 212 is an excellent grout for pumping...even at high flow. For pump recommendations, contact Technical Service. Wet cure for a minimum of 3 days or apply a curing compound which complies with ASTM C-309 on exposed surfaces.

LIMITATIONS

- ▲ Minimum ambient and substrate temperature 45F and rising at time of application.
- ▲ Minimum application thickness: 1/2 in.
- ▲ Do not use as a patching mortar or in unconfined areas.
- ▲ Material must be placed within 15 minutes of mixing.

CAUTION

IRRITANT

Suspect carcinogen - contains portland cement and crystalline silica. Skin and eye irritant. Avoid breathing dust. Use only with adequate ventilation. May cause delayed lung injury (silicosis). IARC lists crystalline silica as having sufficient evidence of carcinogenicity in laboratory animals and limited evidence of carcinogenicity in humans. NTP also lists crystalline silica as a suspect carcinogen. Use of safety goggles and chemical resistant gloves is recommended. In case of high dust concentrations or exceedance of PELs, use an appropriate NIOSH/MSHA approved respirator. Remove contaminated clothing.

FIRST AID

In case of skin contact, wash thoroughly with soap and water. For eye contact, flush immediately with plenty of water for at least 15 minutes; contact physician immediately. Wash clothing before re-use.

CLEAN UP

In case of spillage, ventilate area of spill, confine spill, vacuum or scoop into appropriate container. Dispose of in accordance with current applicable local, state and federal regulations. Uncured material can be removed with water. Cured material can only be removed mechanically.

P.roduct Code 525. Sika and SikaGrout are re
trademarks. Made in USA. Printed in USA. January

KEEP CONTAINER TIGHTLY CLOSED **KEEP OUT OF REACH OF CHILDRE**
NOT FOR INTERNAL CONSUMPTION **FOR INDUSTRIAL USE ONL**
CONSULT MATERIAL SAFETY DATA SHEET FOR MORE INFORMATION

1-800-933-SIKA NATIONWIDE
Regional Information and Sales Centers
For the location of your nearest Sika sales office, contact your regional center.

USA Headquarters
201 Polito Avenue
Lyndhurst, NJ 07071
Phone: 201-933-8800
Fax: 201-933-6225

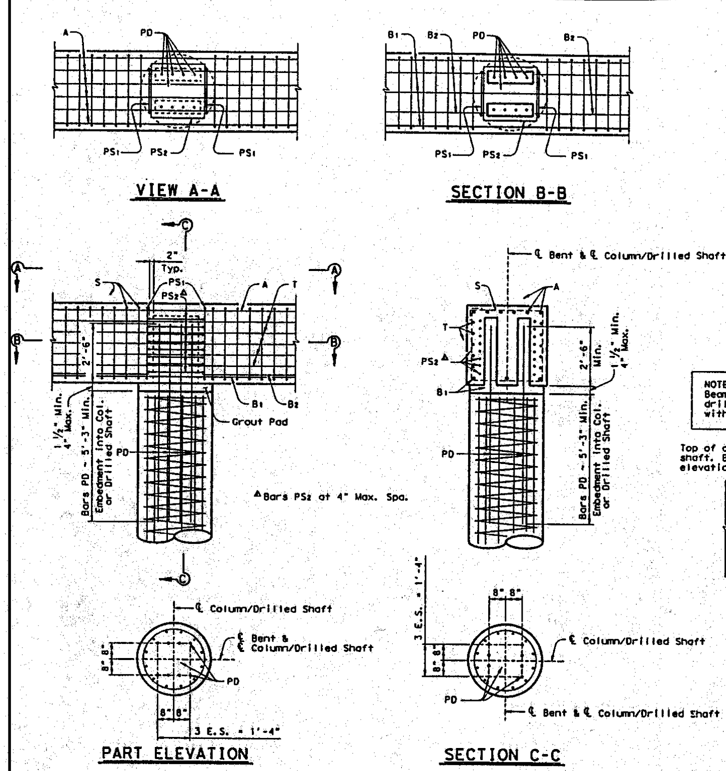
Eastern Region
14 Summit Place
Suite 201
Branford, CT 06405
Phone: 203-488-7706
Fax: 203-488-7790

Central Region
2190 Gladstone Court
Suite A
Glendale Heights, IL 60139
Phone: 630-924-7900
Fax: 630-924-8508

Western Region
12767 East Imperial Hwy.
Santa Fe Springs, CA 90670
Phone: 562-941-0231
Fax: 562-941-4762

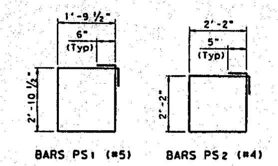
APPENDIX B

PLAN SHEETS FOR EAST FORK TRINITY RIVER BRIDGES

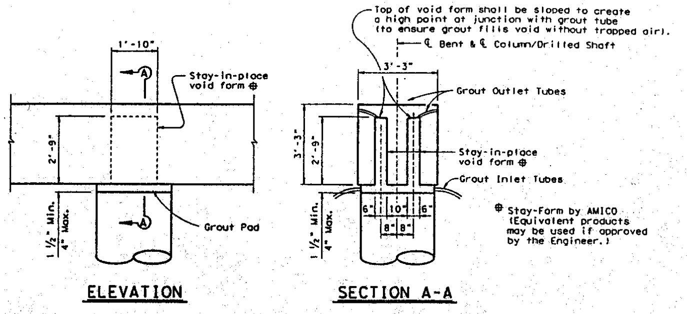


DETAILS SHOWING GROUT POCKET REINFORCING

SHOWING CENTER COLUMN, OUTSIDE COLUMNS SIMILAR



BARS PS1 (#5) BARS PS2 (#4)

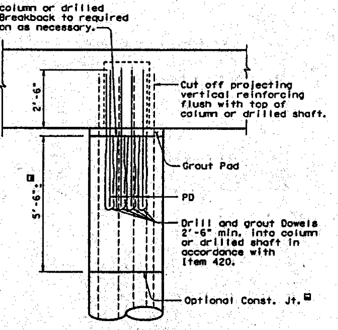


⊕ Stay-Form by AMICO (Equivalent products may be used if approved by the Engineer.)

DETAILS SHOWING GROUT POCKET DIMENSIONS

SHOWING CENTER COLUMN, OUTSIDE COLUMNS SIMILAR

NOTE:
Beam Dowels may be precast into the cap or drilled and grouted into the cap in accordance with Item 420.



DETAILS FOR CONNECTING TO EXISTING COLUMN OR DRILLED SHAFT

SHOWING CENTER COLUMN, OUTSIDE COLUMNS SIMILAR

The Contractor with approval of the Engineer may alternately breakback column or drilled shaft and form/cast the top 5'-6", provided that all the existing steel can be extended into this construction. Projecting vertical steel shall be cut off 2" below top of column or drilled shaft.

PRECAST BENT CONSTRUCTION SEQUENCE

1. Place concrete in accordance with the pertinent requirements of Items 420 and 424. Void forms shall be Stay-Forms by AMICO or equivalent and will be approved by the Engineer.
2. The caps may be moved for storage or placement after the curing has been completed and after a flexural strength of 425 psi (or 3000 psi compression) has been achieved. The Contractor may alter mix design and move caps earlier provided a flexural strength of 425 psi (or 3000 psi compression) has been achieved and provided that the curing time is interrupted by no more than two hours. Lifting locations are indicated in the plans. Stacking shall not exceed three caps high. The Contractor shall store and handle caps in accordance with Item 424 and will be responsible for any damage.
3. Place column/drilled shafts in accordance with Item 420. Top of column/drilled shaft elevations are provided in the plans. If the connection dowels (Bars PD) are inserted after the concrete has been placed, the concrete shall be re-vibrated. Dowel placement tolerance is ± ½" (plan and elevation).
4. Caps may be placed on columns/drilled shafts after the column concrete curing has been completed and after a flexural strength of 425 psi (or 3000 psi compression) has been achieved. Use plastic shims to support the caps at the proper elevation prior to grouting.
5. Grout forms and tubes (input and vent tube type and location) shall be approved prior to grouting. Grout shall be in accordance with the Special Specification "Precast Connection". The grout placement procedures shall be approved by the Engineer. Grout samples will be collected and tested at each bent. Grout tubes shall be cut off flush after grout has set up.
6. Prestressed beams may be placed on the caps after a compressive grout strength of 3000 psi has been achieved.

GENERAL NOTES

Designed according to AASHTO 1996 Standard and current Interim Specifications, and TxDOT Research Project 1748.
All reinforcing shall be Grade 60.
Class "C" Concrete strength f'c = 3600 psi.
All grout shall have compressive strength of 3600 psi according to manufacturer specifications.
See Interior Bent sheets for additional details.

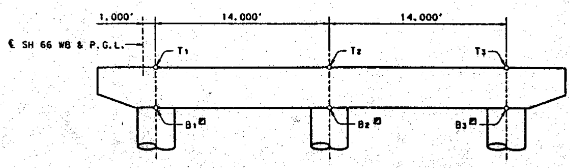
CHANGE ORDER NO.______________________

HS20 LOADING SHEET 1 OF 3

Texas Department of Transportation
Design Division (Bridge)

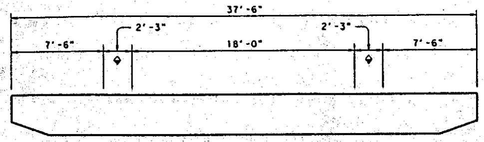
PRECAST CONCRETE
BENT CAP DETAILS
EAST FORK TRINITY
RIVER BRIDGES

FILE: 56921b01.dgn	DN: MH/LW	CK:	DW: RNP	CK: MDH	
©TxDOT AUGUST 2000	DISTRICT	FEDERAL AID PROJECT			SHEET
REVISIONS 8/00 - Precast Bent Cap option details. MDH/RNP	18	NH 99(367)			2490
	COUNTY	CONTROL	SECT	JOB	HIGHWAY
	ROCKWALL	0009	04	039	SH 66



T = Top of Cap Elevations
B = Top of Column or top of Drilled Shaft elevations

ELEVATION



◆ Allowable Lift Point Regions. Design lift point load is 33 tons, including impact factor of 2. If bearing seats are precast, lifting anchors are not allowed at the locations of bearing seats. The Contractor may submit other lifting points/schemes to the Engineer for review and approval. After bent caps are in place, lifting device pockets at expansion joint bents shall be patched with a cement grout.

LIFTING DIAGRAM

EASTBOUND LANES

PRECAST BENT CAP QUANTITY ADJUSTMENTS ◆

Shorten column Bars V by 2'-6" and reduce reinforcing steel total by:
Bents 3 - 5 ~ 387 Lbs.
Bents 6 - 31 ~ 255 Lbs.

Shorten drilled shaft vertical reinforcing by 2'-6" at Bents 2 & 32-44

Lengthen Bars B₂ to 11'-11" (11" additional) ~ 20 Lbs. total

Add 24 Bars PD (#11 X 8'-0") ~ 1020 Lbs. total

Add 6 Bars PS₁ (#5 X 10'-4") ~ 65 Lbs. total

Add 24 Bars PS₂ (#4 X 9'-6") ~ 152 Lbs. total

Reduce Class "C" Concrete by 0.7 C.Y.

Stay in place forms, non shrink grout and grout tubes shall be subsidiary to Class "C" Concrete.

For Contractors information, grout quantity is 0.69 C.Y. per bent.

◆ No changes in pay quantities will be made if Precast Bent Cap option is selected.

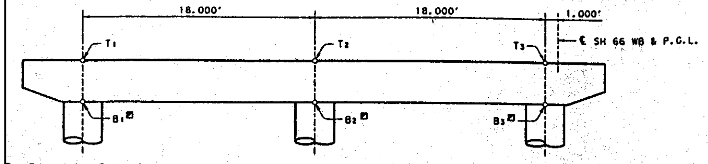
CHANGE ORDER NO.________________

HS20 LOADING SHEET 2 OF 3

Texas Department of Transportation
Design Division (Bridge)

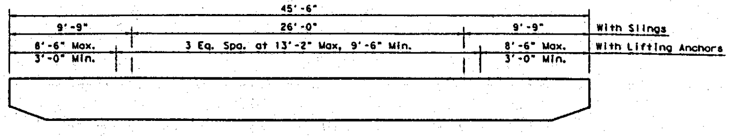
PRECAST CONCRETE
BENT CAP DETAILS
EAST FORK TRINITY
RIVER BRIDGES

FILE: S6921b01.dgn	DN: WH/LW	CK:		DW: RNP		CK: MDH	
© TxDOT AUGUST 2000	DISTRICT	FEDERAL AID PROJECT					SHEET
REVISIONS	18	NH 99(367)					249C
8/00 - Precast Bent Cap option details. MDH/RNP	COUNTY		CONTROL	SECT	JOB		HIGHWAY
	ROCKWALL		0009	04	039		SH 66



T = Top of Cap Elevations
B = Top of Column or top
of Drilled Shaft elevations

ELEVATION



Allowable Lift Points. Design lift point load is 21 Tons with lift anchors and 41 Tons with slings, including impact factor of 2. If bearing seats are precast, lifting anchors are not allowed at the locations of bearing seats. The Contractor may submit other lifting points/schemes to the Engineer for review and approval. After bent caps are in place, lifting device pockets at expansion joint bents shall be patched with a cement grout.

LIFTING DIAGRAM

WESTBOUND LANES

PRECAST BENT CAP QUANTITY ADJUSTMENTS ♦

Shorten column Bars V by 2'-6" and reduce reinforcing steel total by:
Bents 3 & 4 ~ 387 Lbs.
Bents 5 - 34 ~ 255 Lbs.

Shorten drilled shaft vertical reinforcing by 2'-6" at Bents 2 & 35-103

Lengthen Bars B2 to 15'-11" (11" additional) ~ 20 Lbs. total

Add 24 Bars PD (#11 X 8'-0") ~ 1020 Lbs. total

Add 6 Bars PS1 (#5 X 10'-4") ~ 65 Lbs. total

Add 24 Bars PS2 (#4 X 9'-6") ~ 152 Lbs. total

Reduce Class "C" Concrete by 0.6 C.Y.

Stay in place forms, non shrink grout and grout tubes shall be subsidiary to Class "C" Concrete.

For Contractors information, grout quantity is 0.69 C.Y. per bent.

♦ No changes in pay quantities will be made if Precast Bent Cap option is selected.

◇ Top of Cap Elevations are consistent with Bearing Seat Elevations indicated elsewhere in the plans. Bearing seats may be precast or cast-in-place. Any elevation differences shall be taken up in the range of allowable bedding layer thickness and/or bearing seat buildup.

CHANGE ORDER NO.________________

HS20 LOADING SHEET 3 OF 3

Texas Department of Transportation
Design Division (Bridge)

PRECAST CONCRETE
BENT CAP DETAILS
EAST FORK TRINITY
RIVER BRIDGES

FILE: 5692ib01.dgn | DN: WH/LW | CK: | DW: RNP | CK: WDH
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REVISIONS
8/00 - Precast Bent Cap option details. WDH/RNP

DISTRICT	FEDERAL AID PROJECT			SHEET
18	NH 99(367)			2490
COUNTY	CONTROL	SECT	JOB	HIGHWAY
ROCKWALL	0009	04	039	SH 66