REPORT ON PRECAST BEAM CAP PILOT PROJECT

Prepared by Joseph Alderson, Senior Structural Designer
November 2005

Bridge Division
Shyam Gupta, State Bridge Engineer

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I. Introduction

In an effort to use the latest construction techniques, a decision was made to implement a pilot project using precast beam caps rather than traditional cast-in-place techniques. This project was selected to receive Federal funds as part of the Innovative Bridge Construction Program. The goals of the pilot project were fourfold; which were reduced maintenance costs, reduced construction time, increased safety and development of engineering design criteria and new construction methods. Using precast elements meets three of these goals. Precasting should provide a better quality product than cast-in-place methods, which should reduce future maintenance costs. Construction time is reduced with precast elements because they reduce forming time and eliminate curing time during the on-site construction. This can reduce the time that the public is impacted by construction. On many sites, reducing the impact to the public increases safety for the public and workers by limiting conflicts inherent with any road construction. With reduced forming, safety is also increased by reducing the number of workers operating at dangerous heights and reduces the time they have to work at these heights. This was the first time using precast beam caps in the state of Missouri, so design and construction methods were developed as part of this project.

The criteria for the selection of the structure to utilize precast beam caps were small number of bents, columns on pile footings and a smaller structure. Structures with more bents would benefit the most from precast beam caps, but a smaller structure and fewer bents were criteria to limit losses if problems did occur. Columns on footings reduce the uncertainty of the final top of column elevations. Bridge Number A6941 in Franklin County was selected for the pilot project. The bridge carries Route T over Fiddle Creek and was part of Missouri Department of Transportation (MoDOT) job number J6S1056. The bridge is a three-span (43'-47'-43') prestressed concrete I-girder structure. The roadway width is 40 feet. The bridge is skewed 15 degrees and is located on a superelevated horizontal curve. Traffic was maintained on the existing bridge during construction.

To evaluate how well the pilot project met its goals and to continue the development of design and construction procedures for precast beam caps, I was assigned to track the progress of the project as it was constructed. I made site visits to the fabrication plant and the construction site to observe the process and receive feedback from those involved with the process. A partial list of people involved with the project from MoDOT and from the contractors involved is included in the Appendix. This report will highlight the design and construction of this structure. It will also document advantages, disadvantages, problems encountered and recommendations to assist in evaluating the possible use of precast beam caps on future projects.

II. Design and Plans Production

The structural design and construction details closely followed the procedures and reference plans from the Texas Department of Transportation (TxDOT). TxDOT has developed a design methodology for a precast beam cap system, which is included in the Appendix. The main difference in traditional intermediate bent design is that the column-beam connection is analyzed as a fixed connection and then analyzed as a pinned connection rather than just analyzed as a fixed connection. The connection is assumed to be less rigid and functions somewhere between the two extremes. Additionally, the column-to-beam connection is designed. This was done by following the TxDOT procedure and included determining minimum grout strength, embedment depth and confinement reinforcement. The location of the lifting loops was also determined. These design requirements did increase the design time, but not greatly and were not difficult. Typically, on a superelevated roadway the bottom of the beam cap is sloped to reduce the height of the beam on one end. With a precast beam cap, this did not seem practical because of the column-beam connection; so the bottom of the beam was flat. The beam was 3'-6" in height on one end and slightly over 6' on the other. This increased the concrete volume in the cap and correspondingly the weight over a cast-in-place option.

The typical details of the intermediate bent shown on the construction plans were supplemented by details of the column-beam connection, duct and grout tube placement and a lifting diagram. Construction notes were included in the construction plans to provide information on construction procedures, material requirements, testing and payment. A table of grout performance specifications was also provided. The notes and grout specifications adhered to those employed by TxDOT and were obtained from bridge plans for actual projects completed by TxDOT. No bridge special provisions related to the precast beam cap were included. Bridge plan sheets related to the precast beam cap are included in the Appendix. Shop drawings were required for the precast beam cap.

The initial estimate included in the design layout was $519,000 or $82 per square foot, but did not include the precast beam cap option. Another estimate was made assuming $7,000 per precast beam cap with an additional $1,600 for a connection mock up. This increased the cost estimate to $529,000 or $83 per square foot. The Bridge Review Section estimated the cost of each precast beam cap as $8,500 using cost based estimating. An estimate based on substructure concrete and reinforcing steel quantities would have been around $15,000 per cap. The actual unit price bids ranged from $23,815 to $33,000 per cap. The working days estimate was not modified for the use of a precast beam cap. A typical working days estimate is five to ten days for each intermediate bent.

III. Preconstruction

The bridge was awarded to Goodwin Brothers Construction on March 9, 2005 as part of the February 2005 letting at a cost of $918,448.72 not including bridge removal costs. This is a cost of $159.35 per square foot and includes a unit price of $24,610.36 for the precast beam caps. These costs were significantly higher than estimated and exceed the typical cost per square foot of a bridge of this type.

I attended a preconstruction conference on March 31, 2005. The bridge was only minimally discussed, but the main concern of the contractor involved a value-engineering proposal to replace the footings on piles with drilled shafts. The issues related to the precast beam cap were the large weight of the cap and the contractor's inexperience with this new construction technique. The beam caps weighed almost 50 tons, so transporting to the site posed a possible problem. Many of the bridges in the area had weight restrictions, but there was one route that everyone anticipated would be feasible. The large weight also could affect the construction equipment selection, since the caps were heavier than the girders. The contractor's inexperience was not presented as a major concern. It was also learned that Egyptian Concrete had been chosen as the fabricator for the precast beam caps.

After discussions with the Structural Project Manager, Chad Daniel, and myself, it was concluded that the value-engineering proposal to change to drilled shafts would not affect the precast beam cap design or the column-beam connection. It was however decided that the contractor's Engineer should evaluate the entire bent design and be responsible for sealing the construction plans. The updated construction plans for one of the bents are included in the Appendix. The value-engineering proposal was implemented without changes to the precast beam or the beam-to-column connection.

IV. Fabrication

Egyptian Concrete in Salem, Illinois fabricated the precast beam caps. During the fabrication process two changes to Bridge's design were requested. The first was to extend the grout tubes to the top of the beam. This change would allow for easier placement of the grout tubes and be easier to keep them in place while concrete was poured. It would also be easier to pour concrete around the grout tubes and reduce the possibility of voids. Based on these advantages as well as the likelihood that it would make the grouting of the column-beam connection easier, this change was agreed to. TxDOT plans were not consistent in their grout tubes. Some grout tubes had a narrower outlet that terminated on the side of the beam, which was our original design, and others showed the grout tubes extending to the top of the beam without a change in diameter. Grout tubes that extend to the top of the beam are the more practical option.

The second change was reducing the height of the spiral confinement reinforcement from 3'-10" to 3'-6". The spiral reinforcement would have interfered with the stirrup placement as originally detailed. The TxDOT procedure requires the spiral reinforcement to be placed between the main longitudinal reinforcement in the top and bottom of the beam. The height of the spiral reinforcement could be reduced and this requirement still met because of the way the top longitudinal steel was stepped. Detailing of the spiral reinforcement should show the reinforcement extending only to the longitudinal reinforcement and remain below the shear stirrups.

On June 6, 2005 I visited the precaster to observe the pouring of concrete for the beam cap for bent number two. From this visit and discussions with MoDOT material inspectors, I learned additional items to consider for future designs. In addition to the spiral reinforcement issue, the spacing of the longitudinal reinforcement should be given extra consideration. With grout tubes and spiral reinforcement added to a traditional reinforcing bar cage, the spacing of the steel becomes tight. The number of longitudinal bars should be minimized. Also, there should be a note on the construction plans allowing for the shifting of reinforcement to miss grout tubes while maintaining a minimum clearance and spacing. The fabricator did shift the reinforcing bars, but the minimum spacing between bars did not look sufficient. Figures 4.1 and 4.2 show how tight the bar spacing was.

There were some problems that occurred during the concrete placement. It took a longer time to get the forms ready than was expected. A bucket was used to place concrete and only internal vibration was used. Figure 4.3 shows concrete being placed with a bucket. The MoDOT inspector suggested external vibration would be beneficial. The specified slump was eight inches maximum with six percent air content. The mix design was such that these limits were approached. The ambient temperature was 85 degrees Fahrenheit and the concrete temperature was 83 degrees Fahrenheit. Each cap was approximately 24 cubic yards of concrete in volume. The concrete for the precast beam cap was the same as is specified for a cast-in-place beam cap. The precaster used a mix design that they used for other precast elements. During the pour, the rate at which concrete was placed was slow and the 30-minute time limit between trucks was often approached. At one point concrete started to leak out of the bottom of the form on the higher end of the beam cap. The support of the forms had to be increased after the pour had started, and deflection of the forms was a concern throughout. Figure 4.4 shows the formwork and the formwork being reinforced. All of these items increased the time of the pour, as did a rain shower. The problems that occurred can largely be attributed to the fact that Egyptian concrete had no experience with precasting beam caps and the formwork and precast bed were set up from scratch.

Other factors should be considered besides the fabricators inexperience. Due to their limited use, precast beam caps would be considered a specialty item unless their use becomes widespread. This reduces competition and number of bidders, which increases cost and can increase fabrication time. The weight of the caps was also a factor in the fabrication. Not all fabricators would have the resources to move the beam caps in their yards.

Overall the fabricator was pleased to have the work and inquired if more precast beam caps would be utilized in the future. The MoDOT materials inspector felt the "end product came out fairly well." Increased use of precast beam caps should eliminate the problems I witnessed and reduce costs. Only the minor changes to the grout tubes and spiral reinforcement were recommended for future designs. The finished beam cap is shown in Figure 4.5.


Figure 4.1 Rebar Cage and Grout Tubes

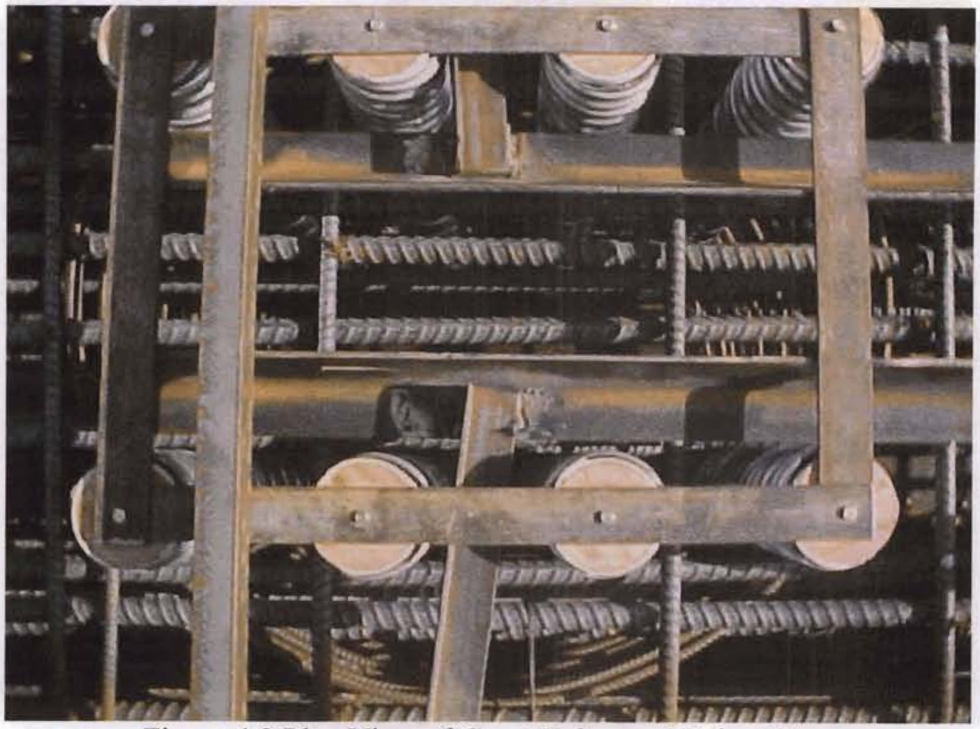

Figure 4.2 Plan View of Grout Tubes and Rebar Cage

Figure 4.3 Placing Concrete by Crane and Bucket

Figure 4.4 Reinforcing Formwork

Figure 4.5 Finished Beam Cap

V. Erection of Beam Caps

I visited the construction site on August 5, 2005 to observe the erection of the beam cap for intermediate bent number three. The cap had been delivered by tractor-trailer, as is shown in Figure 5.1. There was a significant delay before the cap could be set into place due to a miscommunication in the number of lifting loops. The contractor assumed there were two lifting loops, but there were actually four. The cap could not be set until additional cables could be delivered, which resulted in over a two-hour delay. Prior to placing the beam cap, plastic shims were placed on top of the column to ensure the cap would be level and at the proper elevation. Figure 5.2 shows the elevations being measured at the top of the shims. Once the cables were delivered, the placement of the cap went pretty smoothly and quickly. A crane operator and three workers on the ground controlled the placement of the cap via ropes tethered to each end of the cap as shown in Figure 5.3. The actual placement of the cap took approximately ten minutes. The only problem that occurred was that the concrete cracked and broke out around one of the lifting loops on the high side of the cap. Figure 5.4 shows the extent of the concrete that broke out. This area would be repaired when the holes for the lifting loops were filled and an intermediate concrete diaphragm would cover the damaged area. The location of the lifting loops should be more carefully considered in future designs. The lifting loop design should also consider that there is a horizontal component to the lifting force, which could cause cracking or worse if close to a vertical edge in the concrete such as at a beam step.

The beam cap for intermediate bent number two was set in place on August 9, 2005. I did not witness the setting of the beam cap, but was informed by MoDOT personnel that the process went smoothly and quickly. Figure 5.5 shows the beam cap at bent number two being set. There was not any cracking of the concrete around the lifting loop like that which occurred with the beam cap at bent number three.

Setting the beam caps was a quick process, which did not require workers to work at an elevated height like traditional cast-in-place construction. With required preparation and if both caps had been delivered prior to the setting, they could have been placed in a short time frame and certainly in one day. Figure 5.6 shows the beam cap at bent number two in place after setting. With the caps set, the column-beam connection needed to be formed. When the forming was complete, the connection would be filled with grout pumped in under pressure.


Figure 5.1 Beam Cap Delivered by Tractor Trailer

Figure 5.2 Checking Elevations of Shim Plates

Figure 5.3 Setting Beam Cap at Bent No. 3

Figure 5.4 Breakout of Concrete at Lifting Loop

Figure 5.5 Setting Beam Cap at Bent No. 2

Figure 5.6 Beam Cap in Place after Setting

VI. Grouting Column-Beam Connection

I visited the site again on August 30, 2005 to observe the grouting of the column-beam connection. There was a significant delay between the erection of the beam caps and the grouting procedure. The plans required that pressurized grouting be utilized to limit the possibility of voids. The pressurized grouting requirement was required based on TxDOT procedures. Pressurized grouting was not something Goodwin Brothers had experience with, so they subcontracted with Elastizell of Saint Louis. Apparently there was some difficulty finding a contractor familiar with pressurized grouting, which was the main cause of the delay. In addition to a delay, the need for a subcontractor increased the contractor's cost.

The final adjustments were being performed to the forming when I arrived. Steel collars were used as forms, which are shown in Figure 6.1. In addition to being able to resist the pressure from the grouting process, Elastizell had specified that the forms should be watertight. To verify this, the forms should have been filled with water and been able to hold water for 24 hours. This had not been done, so a quick check was performed to see if the forms were watertight. The wet columns shown in Figure 6.2 indicate that the forms were not watertight; some actually leaked quite significantly. Additional work was performed on the forms to try to improve the water tightness, but they were not rechecked prior to grouting. Masterflow® 928 grout was used and was classified as flowable. The grout was pumped under pressure from the mixer, which is shown in Figure 6.3, through a hose to the top of the beam cap. The hose was attached to a plate bolted to the top of the cap that covered four of the eight grout outlet tubes. The plate had one inlet tube and three valves. A second plate covered the remaining four outlet tubes. The workers connecting the hose to the plate is shown in Figure 6.4. As the grout filled the outlet valves they were closed until the grout tubes were all filled. Grout did leak out of the forms while it was being pumped in. Cloth rags were inserted into the leaks to try to stop them. If that did not work a hydraulic sealing compound was used. Figure 6.5 shows the grout that leaked out and the attempt to plug the leak. Figure 6.6 shows the grout tubes completely filled after the grouting process.

Overall the process went efficiently and all of the column-beam connections were grouted in less than two hours. This time could have been reduced if the forms were watertight and time was not spent sealing the leaks or mixing additional grout to replace that which had leaked out. Both the subcontractor and prime contractor expressed their opinions that pressurized grouting was not a necessity. Their opinion was that with flowable grout and internal vibration voids could be eliminated. It was also suggested that monitoring holes in the forms could be employed to check for voids. Eliminating the requirement would reduce costs, eliminate the necessity for formwork strong enough to resist pressure, and more contractors would have the capability of grouting the column-beam connection. Except for also questioning the necessity of pressurized grouting, the MoDOT construction inspector felt the use of precast beam caps "worked out really well."

Figure 6.1 Form Collars

Figure 6.2 Checking if Forms are Watertight

Figure 6.3 Grout Mixer and Hose to Pump Grout to Beam Cap

Figure 6.4 Attaching Hose to Inlet Valve in Plate

Figure 6.5 Grout Leakage and Sealing Leaks

Figure 6.6 Filled Grout Tubes after Grouting Process

VII. Conclusions

Based on the initial goals it can be concluded that the pilot project was a success. The most important goal of developing engineering design criteria and construction methods was met. It can also be concluded that workers' safety can be improved. It was evident that fewer workers would need to work at elevated heights for a shorter duration. The goal of reducing construction time was largely not realized on this project due to the inexperience with using precast beam caps. However, it can be concluded construction time can be reduced with increased use of this option and the experience gained with increased use. This pilot project was not the best project to realize the potential construction time savings that using precast beam caps could provide. A project with a greater number of bents would have better exploited this advantage. It is too early to conclude if the goal of reduced maintenance costs will be met.

From this project much was learned about the design and construction of precast beam caps. The project was thoroughly documented and many more pictures were taken than are included in this report. Video footage was also taken to document the construction. The design of the beams themselves was largely unchanged from a traditional cast-in-place beam cap. Following the procedures and details developed by TxDOT for the additional design requirements of the beam-column connection worked well. The fabricator recommended only minor changes to the reinforcing details and the grout tubes. Extra consideration needs to be given to the bar spacing and lifting loop design. Additional notes or the addition of a bridge special provision may have avoided some of the problems encountered. New construction techniques were developed or existing techniques were employed in new ways. The fabricator had to develop forms and a precasting bed from scratch. The contractor had to find a way to set the beam caps and keep them level and at the correct elevation. The contractor also had to be able to move a large load and set it in a precise position. The pressurized grouting presented a unique challenge, which required the contractor to employ a subcontractor. Formwork that was watertight and able to withstand the pressures created by the grouting process had to be developed. A method to pump the grout through an inlet tube and ensure all outlet tubes were completely full had to be created. The subcontractor used their expertise in a new way on this project. The contractor, subcontractor and MoDOT construction inspector were able to suggest a change to the design requirement for pressurized grouting based on this experience and develop their conclusion that it was an unnecessary expense and delay. Future use of precast beam caps should go more smoothly and quickly based on the experience gained and lessons learned from this pilot project.

Worker safety can be improved by utilizing precast beam caps in construction. It can be assumed that four or more workers would be required to perform elevated work for several days with cast-in-place construction. Forming and pouring concrete for the beam cap are eliminated through the use of precast beam caps. No workers were required to perform elevated work during the setting of the beam caps. Two workers on top of the beam cap seemed adequate during the grouting process. The time spent by those two workers grouting was less than would be required for forming, pouring concrete and stripping forms. Using precast beam caps does not eliminate the necessity of elevated work, but it does reduce it.

Unfortunately lack of experience using precast beam caps mostly eliminated the reduction in construction time that is anticipated from using prefabricated elements. The project did demonstrate that construction time could be saved if not for the delays caused by inexperience and the necessity to develop new construction methods. Actually setting the caps and grouting the beam-column connection took very little time. Approximately ten minutes were required to set the cap in place and grouting both caps took about two hours. Even with a small number of bents, it can be seen that time savings could be realized. This time savings would be magnified for structures with a greater number of bents. Any reduction in construction time has added benefits of reducing the impact on the public and reducing the time construction and traffic are in conflict, which increases safety for both workers and the traveling public.

Although it is too early to tell on this project, this option could provide the advantage of reduced maintenance costs. Constructing the beam caps in a more controlled environment under better conditions should provide a higher quality product. A higher quality product should be less susceptible to cracking and spalling that would need to be repaired. This not only reduces maintenance costs and time, but also increases safety for the traveling public.

Conclusions about cost can be made even though it was not a main goal of this pilot project. The bid price was three times what was estimated. Cost is increased because this is a new option in Missouri. Prefabricators consider precast beam caps a specialty item, so they will charge more. The contractor most likely increased the bid price due to unfamiliarity with this option and the extra risk involved. Extra time and money had to be spent by the fabricator and contractor developing new construction techniques. The pressurized grouting requirement necessitated a subcontractor that added \$7,000 to the cost according to John Byrd of Goodwin Brothers. Increased usages of the option may reduce costs, but it would need to become more widespread to be competitive with cast-in-place methods.

The pilot project did demonstrate the advantages and disadvantages of the use of precast beam caps. It also was a valuable learning experience for design and construction. Lessons learned should contribute to improving the process and allowing the benefits of this option to be maximized and the negatives to be minimized. The project should also help determine what place this option has in future MoDOT projects.

VIII. Recommendations

From the conclusions reached during the pilot project, recommendations for future use of this option can be made. Selecting which projects where this option should be used is important. The best use of this option would be on projects where time considerations exceed those of cost. However, it should be considered if the inexperience with this option would eliminate any time savings. Projects with a large number of bents are the best possibility where the time saving would be great enough to overcome any delays possible with new construction methods. Structures on superelevated roadways are not good candidates for precast beam caps. Unless the column-beam connection can be adapted for a beam with a sloped bottom, the beam can become massive, which can exacerbate issues dealing with the weight of the caps. Transporting the beam caps needs to be considered. Routes with posted bridges, county roads and residential streets may not be viable or may cause the contractor to incur costs for damaged roads. The footing types should not be a significant concern in structure selection because the cap rests on shims that can account for changes in the top of column elevation. There are situations where precast beam caps would be beneficial, but the circumstances should be fully considered.

The design process does not need major changes. The congestion of the reinforcement cage should be considered and spiral confinement reinforcement should be detailed so it does not interfere with the shear stirrups. The grout tubes should extend to the top of the beam cap and do not require a change in diameter. The design and placement of the lifting loops should consider that the lifting force will not be purely vertical and edge distance to the lifting loops is important to prevent cracking. A note on the plans allowing the shifting of reinforcement to clear the grout tubes should be added. Notes or a special provision clarifying that pressurized grouting will be required could eliminate miscommunication. Notes or a special provision could inform that formwork should be watertight and that it should demonstrate that by holding water for 24 hours prior to grouting. It should also be noted that the water should be removed prior to grouting rather than displaced by pumping the grout in. The estimated costs of the beam caps should take into account that they are a specialty item and an increased risk to the contractor as long as they are used infrequently. Working days should reflect the possibility that delays will occur due to the fabricators' and contractors' inexperience with this type of construction. TxDOT's procedures and plans combined with this project's lessons should provide the necessary information for the Bridge Division to use this option in the future.

Reducing costs would make this option more viable. Widespread use and increased experience gained by fabricators and contractors is one way costs could come down. With increased use more contractors and fabricators will have developed the techniques and acquired the equipment to reduce costs. Also more competition should develop to reduce bid prices. Time savings with this method could also translate into reduced costs. Labor costs would be affected by the time savings and by transferring labor from the field to the fabricator. Presumed maintenance cost reduction should also be considered as part of the overall cost of this option. The process of using precast beam caps should continue to be evaluated for changes in design or construction techniques. The pressurized grouting requirement is one example that should be evaluated. Cost is the main prohibitive factor in limiting the use of this option; lowering the cost through increased use or design changes should be considered to make this option more beneficial.

The precast beam cap pilot project has shown that this option would currently be advantageous under certain narrow conditions. The importance of time versus cost is the main criteria in choosing this option. However, there is potential to reduce the cost of this option with increased use. This option should be considered where reducing construction time is the controlling factor. It should also be evaluated if wider use would have a future benefit that exceeds the current disadvantages of using this option.

VIII. Appendix

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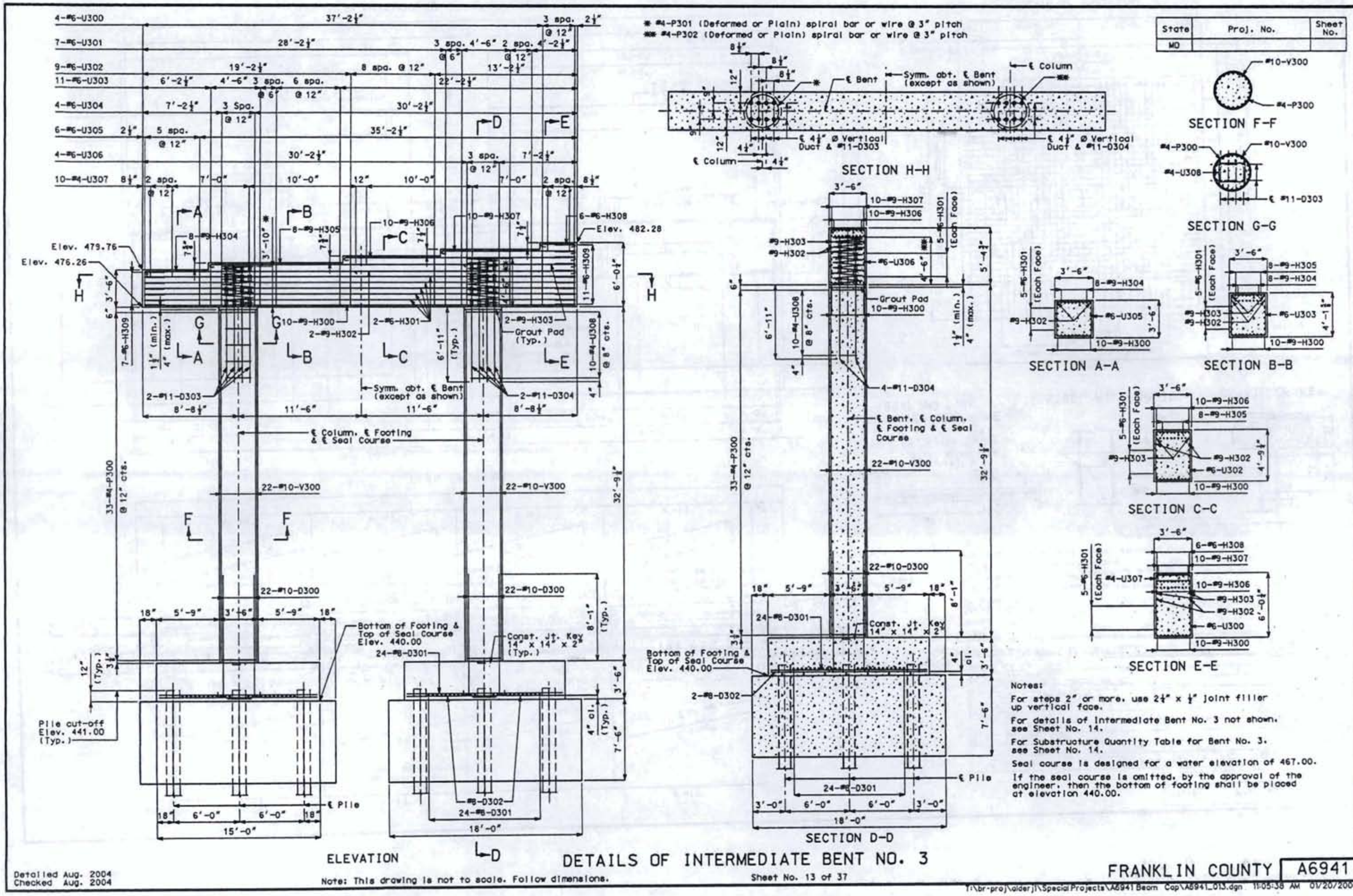
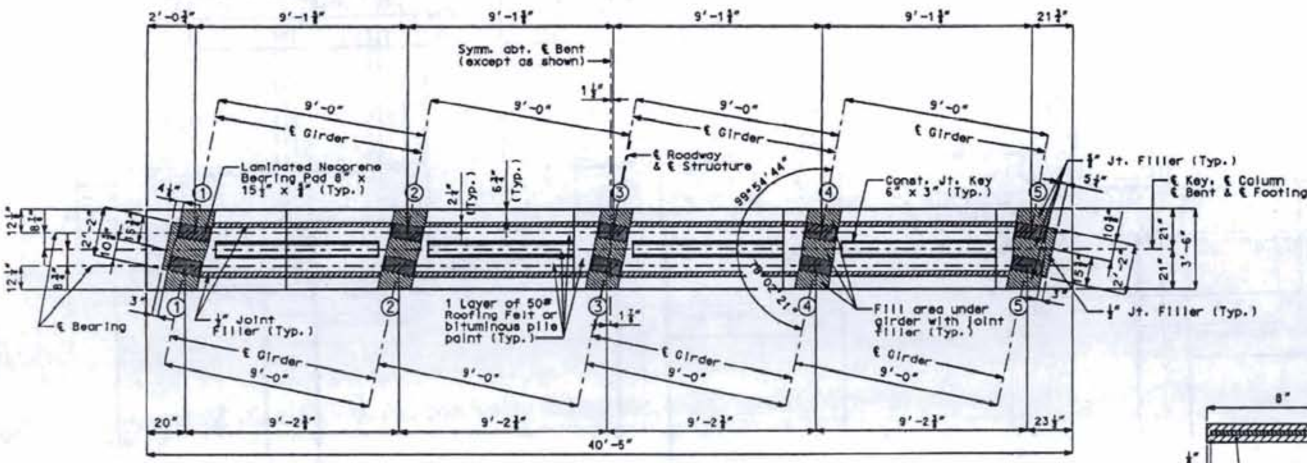


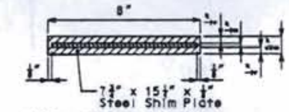
DETAILS OF INTERMEDIATE BENT NO. 3
ELEVATION
SECTION H-H
SECTION F-F
SECTION G-G
SECTION A-A
SECTION B-B
SECTION C-C
SECTION E-E
SECTION D-D
* #4-P301 (Deformed or Plain) spiral bar or wire @ 3" pitch
** #4-P302 (Deformed or Plain) spiral bar or wire @ 3" pitch
Elev. 479.76
Elev. 476.26
Elev. 482.28
Bottom of Footing & Top of Seal Course Elev. 440.00
Pile cut-off Elev. 441.00 (Typ.)
Const. Jt. Key 14" x 14" x 2" (Typ.)
Grout Pad (Typ.)
Symm. abt. ℄ Bent (except as shown)
℄ Column, ℄ Footing & ℄ Seal Course
℄ Bent, ℄ Column, ℄ Footing & ℄ Seal Course
℄ Pile
Notes:
For steps 2" or more, use 2½" x ½" joint filler up vertical face.
For details of Intermediate Bent No. 3 not shown, see Sheet No. 14.
For Substructure Quantity Table for Bent No. 3, see Sheet No. 14.
Seal course is designed for a water elevation of 467.00.
If the seal course is omitted, by the approval of the engineer, then the bottom of footing shall be placed at elevation 440.00.
State
Proj. No.
Sheet No.
MO
Detailed Aug. 2004
Checked Aug. 2004
Note: This drawing is not to scale. Follow dimensions.
Sheet No. 13 of 37
FRANKLIN COUNTY
A6941
REV.

Appendix 3

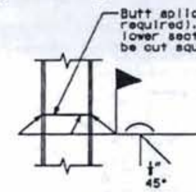
State	Proj. No.	Sheet No.
MO		



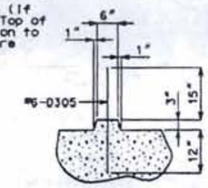
PLAN OF BEAM



TYP. SECTION THRU 8" X 15½" X ½" LAMINATED NEOPRENE BEARING PAD



STEEL PILE SPLICE



DETAIL OF KEY

Substructure Quantity Table for Bent No. 3

Item		Quantity
Class 1 Excavation	cu. yard	110
Class 2 Excavation	cu. yard	684
Cofferdams - Bent 3	lump sum	1
Structural Steel Piles (14 In.)	linear foot	864
Pile Point Reinforcement	each	18
Class B Concrete (Substructure)	cu. yard	81.7
Seal Concrete	cu. yard	180
Pre-Cast Bent Cap	each	1
Reinforcing Steel (Bridges)	pound	14,250

Notes:

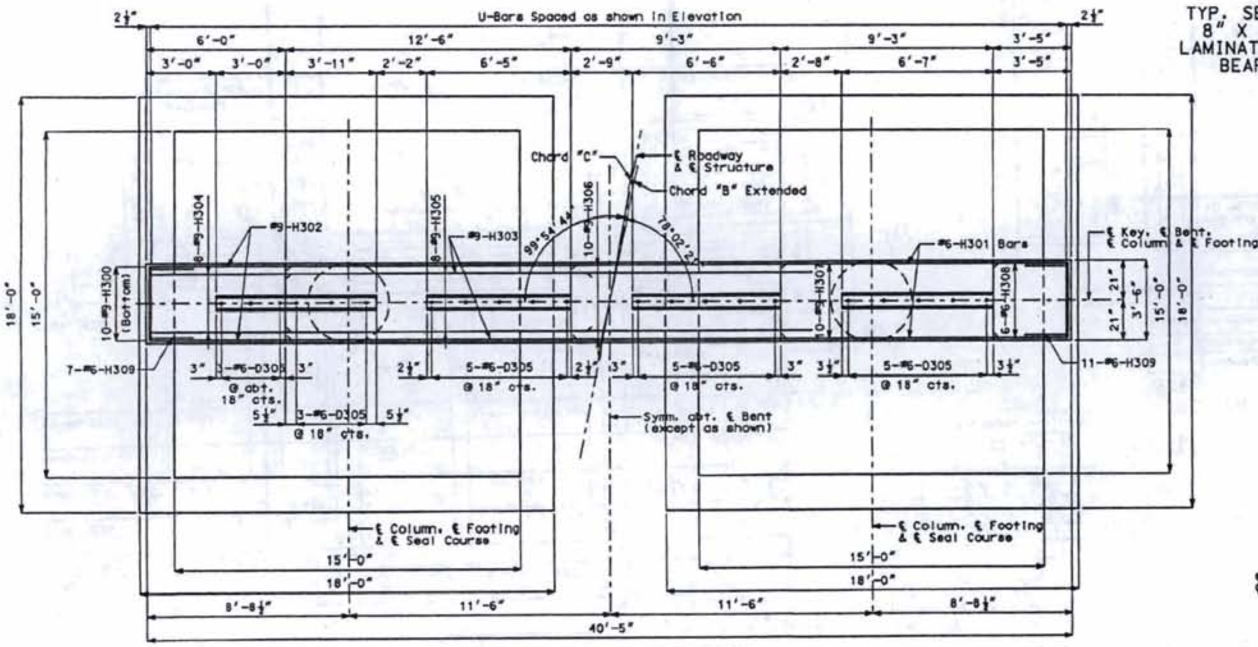
These quantities are included in the Estimated Quantities Table on Sheet No. 2.

For details of Intermediate Bent No. 3 not shown, see Sheet Nos. 13 & 15.

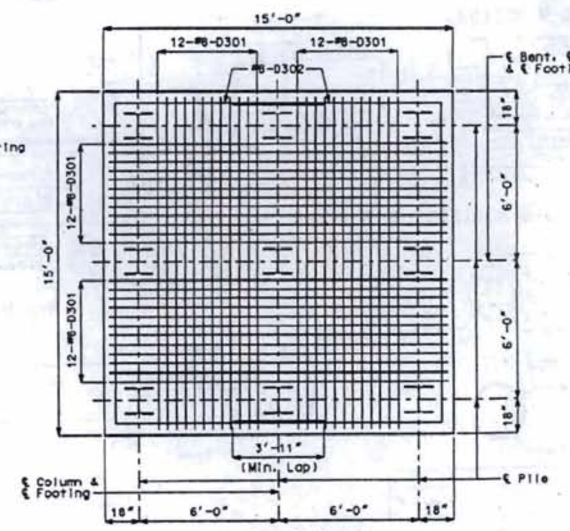
For steps 2" or more, use 2½" x ½" joint filler up vertical face.

For plan showing substructure layout, see Sheet No. 4.

For plan showing prestressed girder & bearing layout, see Sheet No. 5.



PLAN SHOWING REINFORCEMENT



PLAN OF FOOTING SHOWING REINFORCEMENT

DETAILS OF INTERMEDIATE BENT NO. 3

Detailed Aug. 2004
Checked Aug. 2004

Note: This drawing is not to scale. Follow dimensions.

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Appendix 5

PRECAST BENT CAP CONSTRUCTION NOTES:

Construction shall be in accordance with the requirements of Sec 703. Vertical Ducts shall be semi-rigid spirally crimped, corrugated duct of galvanized, cold rolled steel conforming to ASTM A527 and ASTM A619. Corrugations shall have an amplitude of 0.094 in.

Precast Bent Caps shall be handled, moved, stored and placed in the structure in a manner to avoid chipping, cracking, fractures and excessive bending stresses or damage. Precast Caps shall be supported on firm blocking until placed and shimmed into final position. Blocking shall be installed such that uneven settlement due to wet ground or inadequate material underneath the blocking shall not occur.

Place column in accordance with Sec 703. If the connection dowels (D203, D204, D303 & D304) are inserted after the concrete has been placed, the concrete shall be re-vibrated. Dowel placement tolerance is + $\frac{1}{2}$" (plan and elevation).

Caps shall only be placed on columns after the column concrete has achieved a flexural strength of 355 psi (or 2500 psi compressive strength). Use plastic shims or friction collars to support the caps at the proper elevation prior to grouting. Total area of plastic shims used on top of each column shall not exceed 6% of the column area. Column curing may be interrupted a maximum of two hours to allow placement of plastic shims or friction collars and placement of the cap.

Grout forms and tubes (input type and location) shall be approved by the engineer prior to grouting. Connection shall be grout tight such that fluid grout does not leak out before grout has achieved initial set. The contractor is advised that excessive grout pressures may damage grout forms and tubes.

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 the Table of Grout Performance Specifications, including mechanical properties, compatibility, constructability and durability. Table 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. Fluid grout shall not exhibit frothing or foaming.

Prior to construction, the Contractor shall demonstrate the adequacy of the grout and the grouting system meeting the approval from the Engineer.

Two (2) 2" x 2" x 2" grout cube samples shall be cast for each precast bent cap (one per precast cap-to-column connection) by the Contractor and given to MoDOT for testing to ensure that the grout meets the specified mechanical property criteria for compressive strength. Failure to meet strength criteria or evidence of frothing or foaming will be cause for removal and recasting of the connection as deemed necessary by the Engineer. Grout selected shall meet the other criteria listed as well.

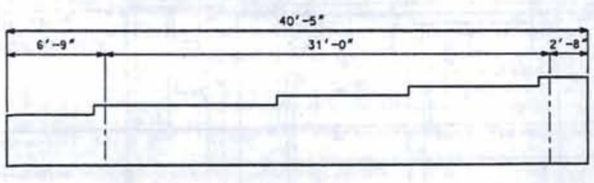
Prestressed beams shall be placed on the caps after a grout compressive strength of 2500 psi has been achieved.

If the Contractor deviates from these requirements, a work plan and supporting calculations shall be submitted to the Engineer for review and approval. The plan and calculations shall be sealed by an Engineer registered in the State of Missouri.

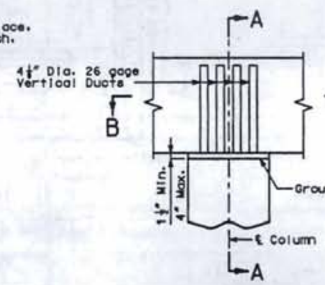
Payment for the work necessary to complete Pre-Cast Bent Caps, including all material, labor, tools, equipment and any other incidental work necessary to complete this item, complete-in-place, will be considered completely covered by the contract unit price for Pre-Cast Bent Cap per each.

TABLE OF GROUT PERFORMANCE SPECIFICATIONS

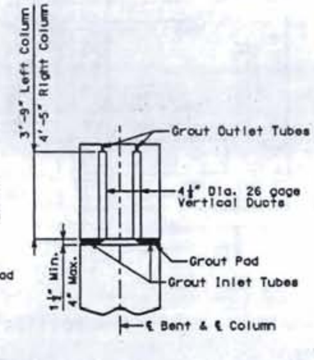
PROPERTY	VALUES	
MECHANICAL	AGE	COMPRESSIVE STRENGTH (psi)
Compressive strength (ASTM C-109, 2" cubes)	1 day	2500
	3 days	4000
	7 days	5000
	28 days	5800
COMPATIBILITY		
Expansion Requirements (ASTM C-827 & ASTM C-1090)	Grade B or C - expansion per ASTM C-1107	
Modulus of elasticity (ASTM C-469)	2.8 - 5.0 X 10⁶ psi	
Coefficient of thermal expansion (ASTM C-531)	3.0 - 10.0 X 10⁶/deg F	
CONSTRUCTABILITY		
Flowability (ASTM C-939; CRD-C 611 Flow Cone)	fluid consistency eflux time: 20-30 seconds	
Set Time (ASTM C-191) Initial	2.5-5.0 hrs	
Final	4.0-8.0 hrs	
DURABILITY		
Freeze Thaw (ASTM C-666)	300 cycles, RDF 90%	



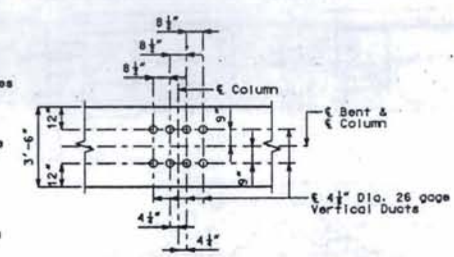
LIFTING DIAGRAM



ELEVATION SHOWING DUCT PLACEMENT



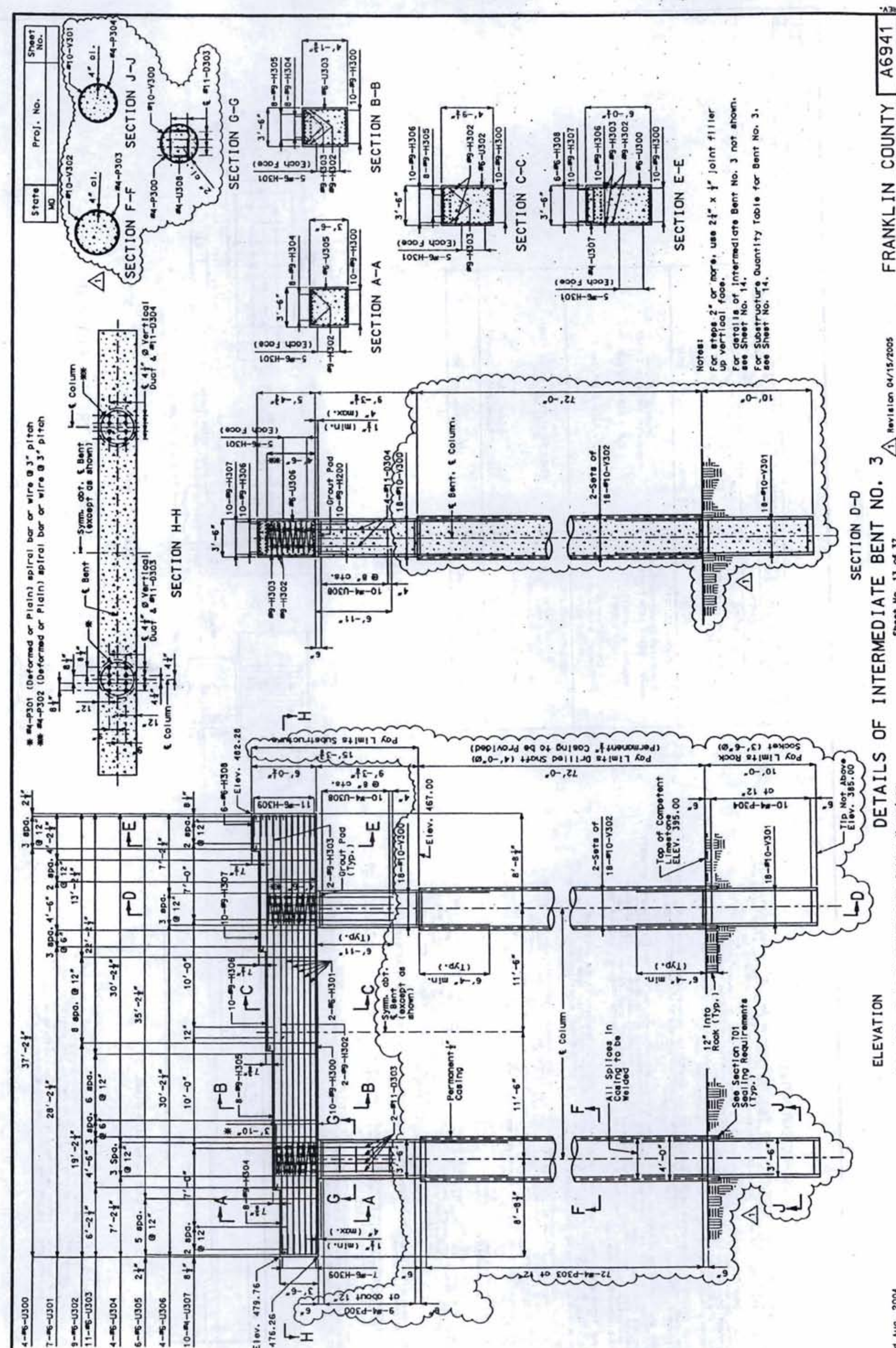
SECTION A-A



SECTION B-B

DETAILS OF PRECAST CONCRETE BENT CAP

Note:
Design Lift Point Load is 49 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 shall be patched with a cement grout.


DETAILS OF INTERMEDIATE BENT NO. 3
FRANKLIN COUNTY
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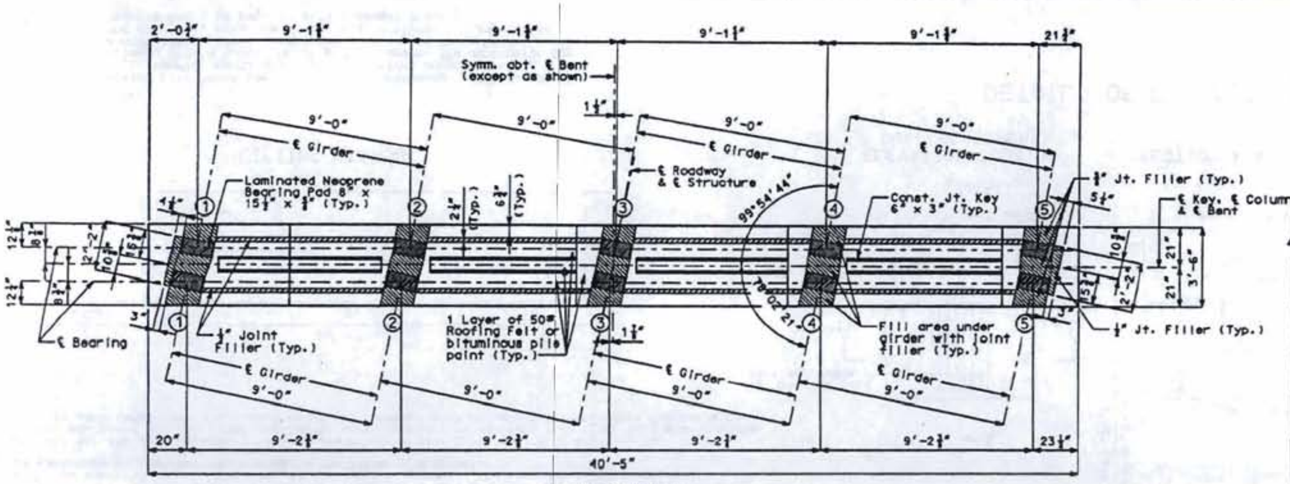
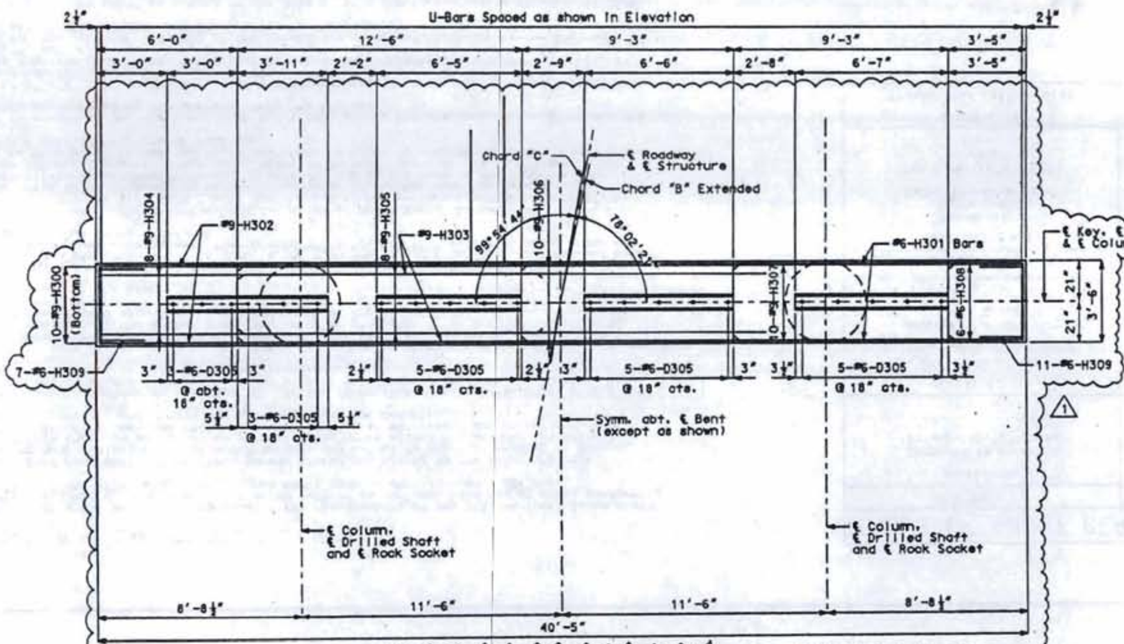
Sheet No. 13 of 37
ELEVATION
SECTION D-D
SECTION H-H
SECTION A-A
SECTION B-B
SECTION C-C
SECTION E-E
SECTION F-F
SECTION G-G
SECTION J-J
Notes:
For steps 2" or more, use 2½" x ½" joint filler up vertical face.
For details of Intermediate Bent No. 3 not shown, see Sheet No. 14.
For Substructure Quantity Table for Bent No. 3, see Sheet No. 14.
Note: This drawing is not to scale. Follow dimensions.
Detailed Aug. 2004
Checked Aug. 2004
Revision 04/15/2005

Appendix 7

State	Proj. No.	Sheet No.
MO		



PLAN OF BEAM



PLAN SHOWING REINFORCEMENT

Substructure Quantity Table for Bent No. 3		
Item		Quantity
Foundation Test Holes	lin. ft.	40
Concrete Coring	lin. ft.	146
Supplementary Television Inspection	each	2
Sonic Logging Testing	each	2
Drilled Shafts (4'-0"Ø)	lin. ft.	144
Rock Sockets (3'-6"Ø)	lin. ft.	20
Class B Concrete (Substructure)	cu. yard	6.6
Pre-Cast Bent Cap	each	1
Reinforcing Steel (Bridges)	pound	19,460

Notes:

These quantities are included in the Estimated Quantities Table on Sheet No. 2.

* All reinforcement in Drilled Shafts and Rock Sockets are included in Substructure Quantities.

The contract unit price for "Drilled Shafts (4'-0"Ø)" shall be full compensation for construction of the drilled shafts including Class B2 Concrete within the pay limits shown on sheet no. 13. The contract price does not include reinforcing steel.

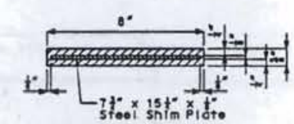
The contract unit price for "Rock Sockets (3'-6"Ø)" shall be full compensation for construction of the rock sockets including Class B2 Concrete within the pay limits shown on sheet no. 13. The contract price does not include reinforcing steel.

For details of Intermediate Bent No. 3 not shown, see Sheet No. 13 & 15.

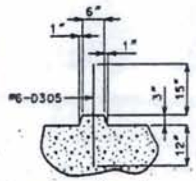
For steps 2" or more, use 2½" x ½" joint filler up vertical face.

For plan showing substructure layout, see Sheet No. 4.

For plan showing prestressed girder & bearing layout, see Sheet No. 5.



TYP. SECTION THRU
8" X 15½" X ½"
LAMINATED NEOPRENE
BEARING PAD



DETAIL OF KEY

DETAILS OF INTERMEDIATE BENT NO. 3

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Checked Aug. 2004

Note: This drawing is not to scale. Follow dimensions.

Sheet No. 14 of 37

Revision 04/15/2005

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State	Proj. No.	Sheet No.
MO		

PRECAST BENT CAP CONSTRUCTION NOTES:

Construction shall be in accordance with the requirements of Sec 703. Vertical Ducts shall be semi-rigid spirally crimped, corrugated duct of galvanized, cold rolled steel conforming to ASTM A527 and ASTM A619. Corrugations shall have an amplitude of 0.094 in.

Precast Bent Caps shall be handled, moved, stored and placed in the structure in a manner to avoid chipping, cracking, fractures and excessive bending stresses or damage. Precast Caps shall be supported on firm blocking until placed and shimmed into final position. Blocking shall be installed such that uneven settlement due to wet ground or inadequate material underneath the blocking shall not occur.

Place column in accordance with Sec 703. If the connection dowels (D203, D204, D303 & D304) are inserted after the concrete has been placed, the concrete shall be re-vibrated. Dowel placement tolerance is + ½" (plan and elevation).

Caps shall only be placed on columns after the column concrete has achieved a flexural strength of 355 psi (or 2500 psi compressive strength). Use plastic shims or friction collars to support the caps at the proper elevation prior to grouting. Total area of plastic shims used on top of each column shall not exceed 6% of the column area. Column curing may be interrupted a maximum of two hours to allow placement of plastic shims or friction collars and placement of the cap.

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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 the Table of Grout Performance Specifications, including mechanical properties, compatibility, constructability and durability. Table 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. Fluid grout shall not exhibit frothing or foaming.

Prior to construction, the Contractor shall demonstrate the adequacy of the grout and the grouting system meeting the approval from the Engineer.

Two (2) 2" x 2" x 2" grout cube samples shall be cast for each precast bent cap (one per precast cap-to-column connection) by the Contractor and given to MoDOT for testing to ensure that the grout meets the specified mechanical property criteria for compressive strength. Failure to meet strength criteria or evidence of frothing or foaming will be cause for removal and recasting of the connection as deemed necessary by the Engineer. Grout selected shall meet the other criteria listed as well.

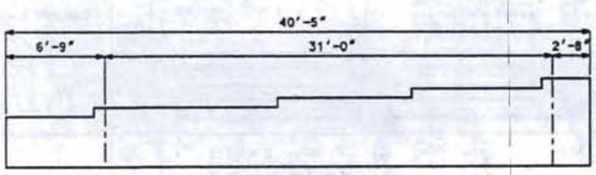
Prestressed beams shall be placed on the caps after a grout compressive strength of 2500 psi has been achieved.

If the Contractor deviates from these requirements, a work plan and supporting calculations shall be submitted to the Engineer for review and approval. The plan and calculations shall be sealed by an Engineer registered in the State of Missouri.

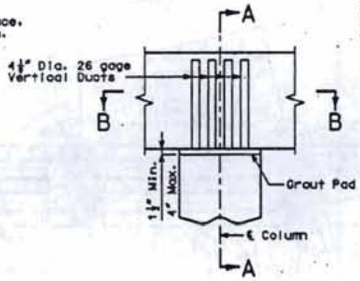
Payment for the work necessary to complete Pre-Cast Bent Caps, including all material, labor, tools, equipment and any other incidental work necessary to complete this item, complete-in-place, will be considered completely covered by the contract unit price for Pre-Cast Bent Cap per each.

TABLE OF GROUT PERFORMANCE SPECIFICATIONS

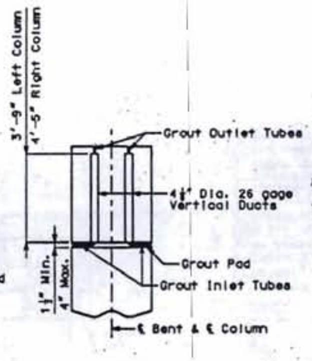
PROPERTY	VALUES	
MECHANICAL Compressive strength (ASTM C-109, 2" cubes)	AGE 1 day 3 days 7 days 28 days	COMPRESSIVE STRENGTH (psi) 2500 4000 5000 5800
COMPATIBILITY Expansion Requirements (ASTM C-827 & ASTM C-1090)	Grade B or C - expansion per ASTM C-1107	
Modulus of elasticity (ASTM C-469)	2.8 - 5.0 X 10^6 psi	
Coefficient of thermal expansion (ASTM C-531)	3.0 - 10.0 X 10^6/deg F	
CONSTRUCTABILITY Flowability (ASTM C-939; CRD-C 611 Flow Cone)	fluid consistency eflux time: 20-30 seconds	
Set Time (ASTM C-191) Initial Final	 2.5-5.0 hrs 4.0-8.0 hrs	
DURABILITY Freeze Thaw (ASTM C-666)	300 cycles, RDF 90%	



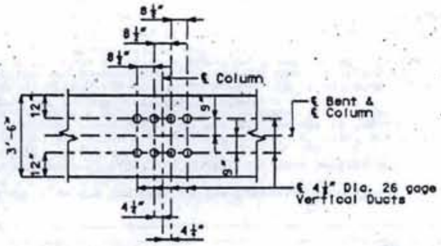
LIFTING DIAGRAM



ELEVATION SHOWING DUCT PLACEMENT



SECTION A-A



SECTION B-B

DETAILS OF PRECAST CONCRETE BENT CAP

Note:

Design Lift Point Load is 49 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 shall be patched with a cement grout.

Detailed Aug. 2004
Checked Aug. 2004

Note: This drawing is not to scale. Follow dimensions.

Sheet No. 15 of 37

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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.

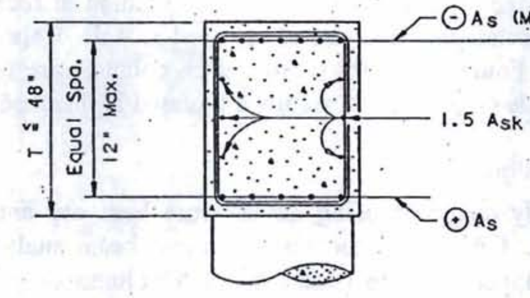


Figure 6.1 Typical Bent Cap Reinforcement—Rectangular Cross Section [6.1]

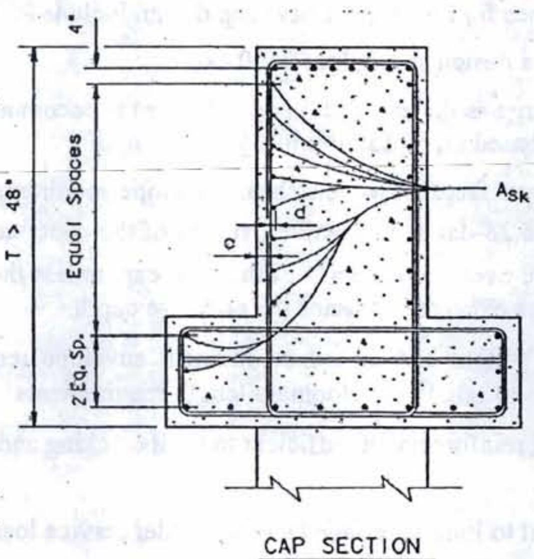
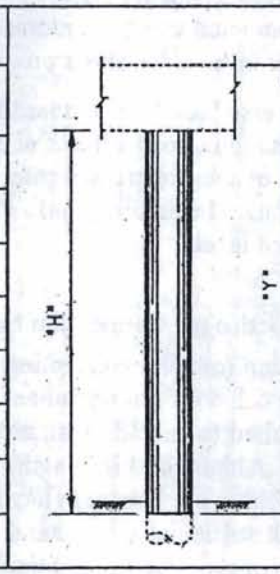


Figure 6.2 Typical Bent Cap Reinforcement—Inverted-tee Cross Section [6.1]

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: No. of Bars	Vertical Steel Bars V: Size	Vertical Steel Bars V: %	Bars Z: Size & Pitch	Height "Y" Maximum
18"ø	6	# 6	1.04	# 3 @ 6"	18'
24"ø	8	# 7	1.06	# 3 @ 6"	24'
30"ø	8	# 9	1.13	# 3 @ 6"	30'
36"ø	10 8	# 9 # 10	0.98 1.00	# 4 @ 9"	36'
42"ø	12	# 10	1.10	# 4 @ 9"	42'
48"ø	18	# 9	0.99	# 4 @ 9"	48'
54"ø	18	# 10	1.00	# 4 @ 9"	54'
60"ø	22	# 10	0.99	# 4 @ 9"	60'



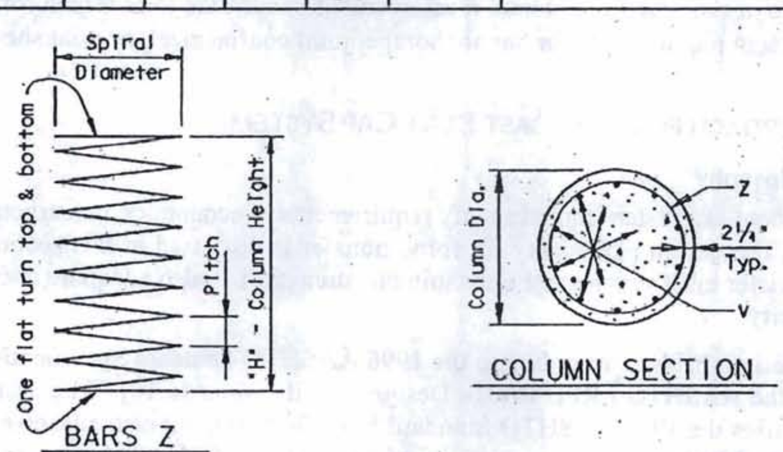


Figure 6.3 TxDOT Standard Column Sizes and Minimum Reinforcement [6.1]

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 <u>not</u> 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.

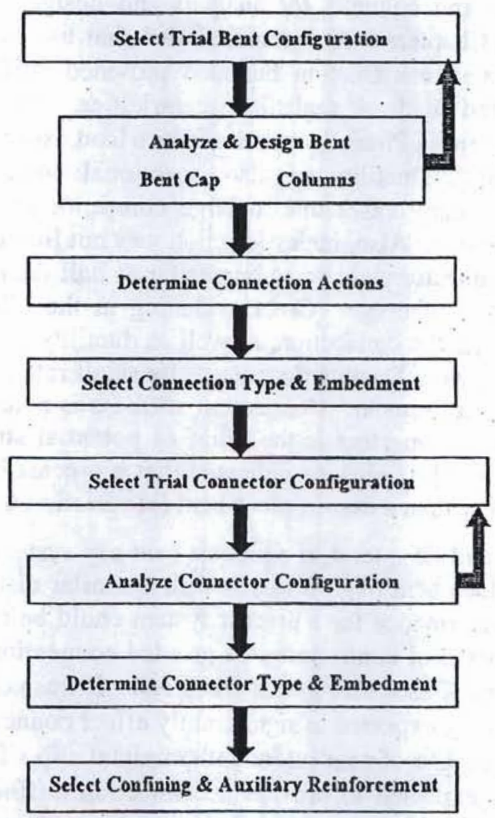


Figure 6.4 Design Flow Chart for Precast Bent Cap System

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.

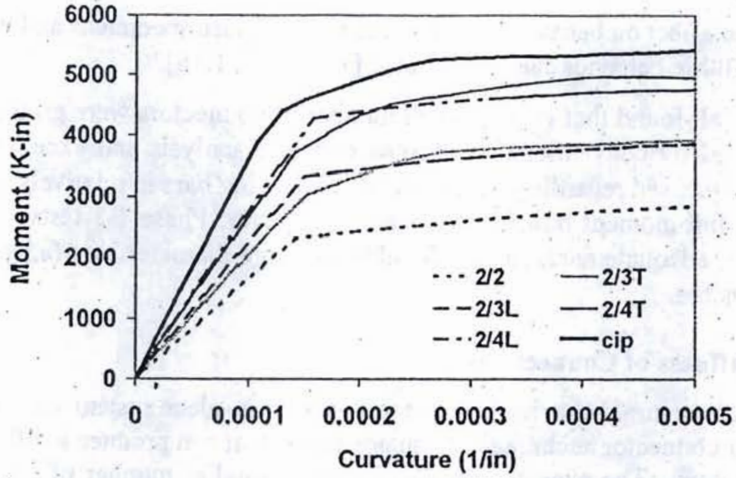


Figure 6.5 Predicted Moment-Curvature Response for Connector Configurations

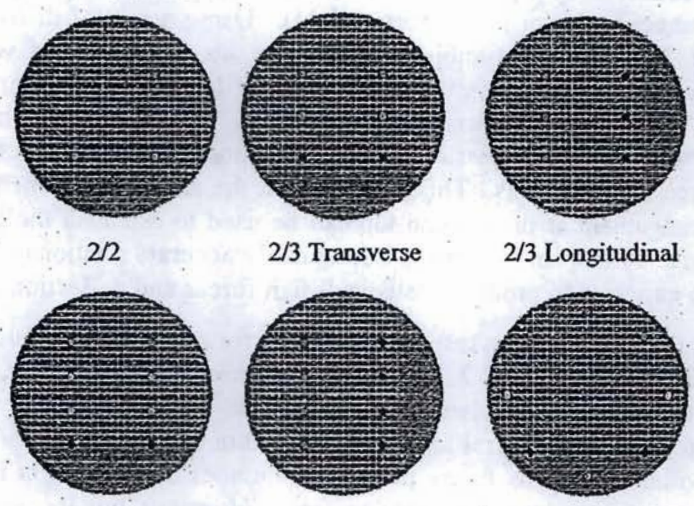


Figure 6.6 Predicted Neutral Axis Locations for Various Connector Configurations

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.

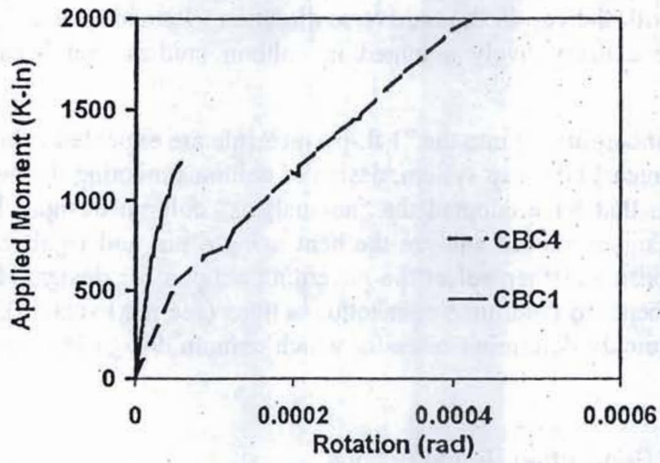


Figure 6.7 Moment-Rotation Relationship for Phase 2 CBC Specimen

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.

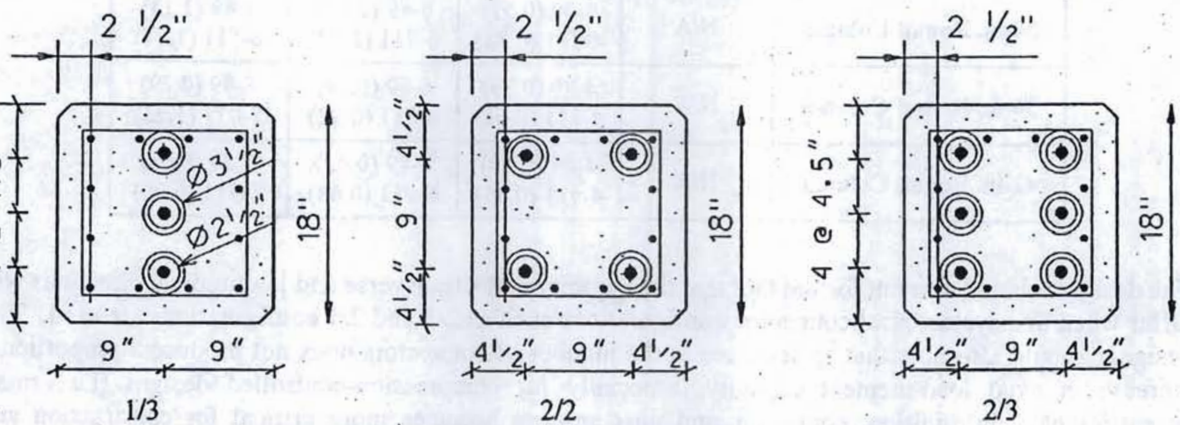


Figure 6.8 Example Connector Configurations—18-in. Pile

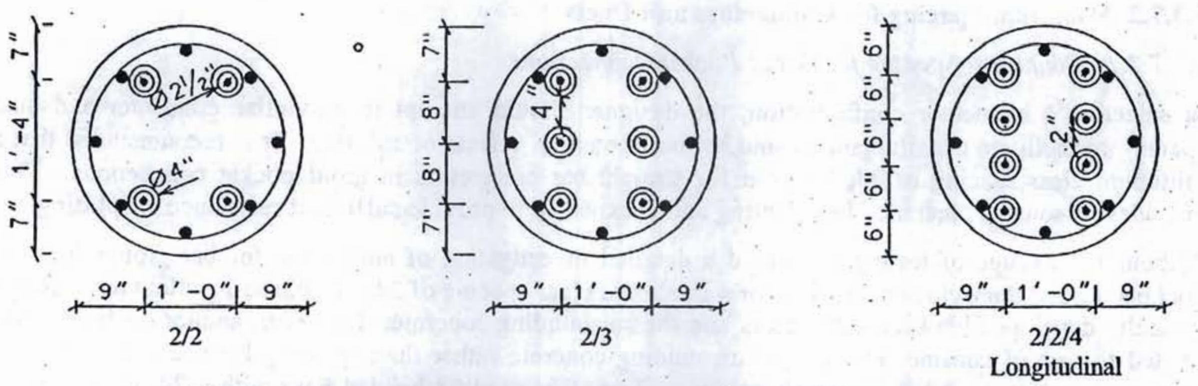


Figure 6.9 Example Connector Configurations—30-in. Column

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).

F~~or 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.

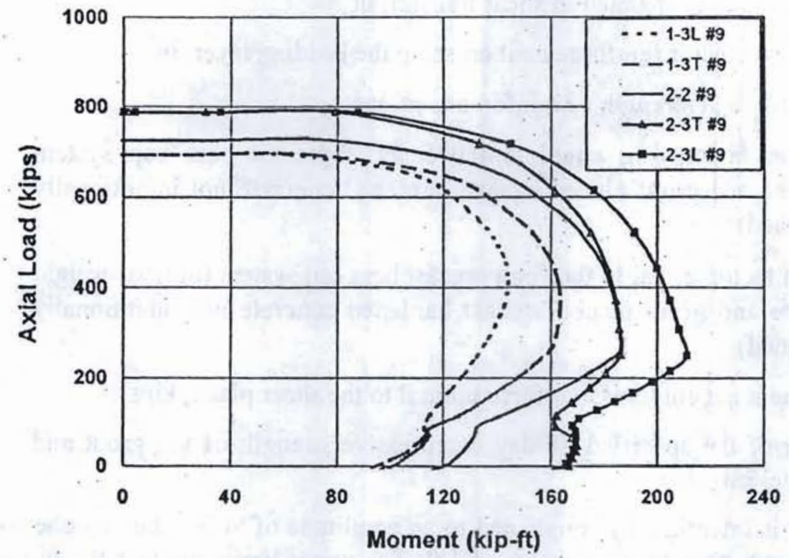


Figure 6.10 Axial Load-Moment Design Strength Interaction Curve—18-in. Pile

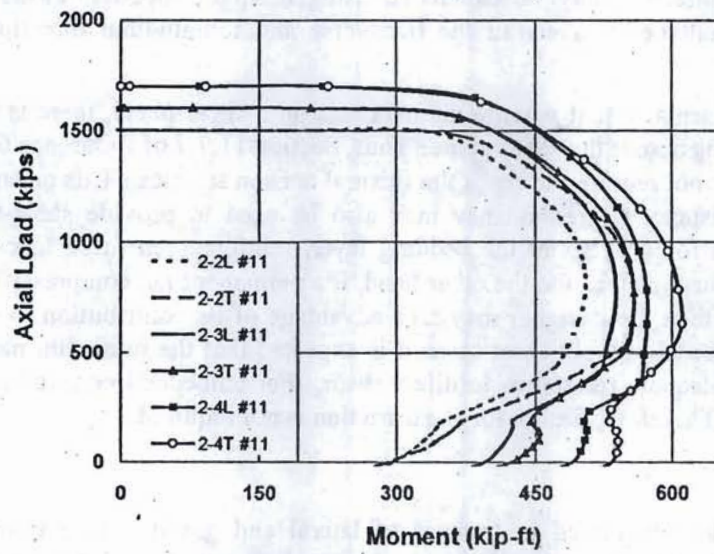


Figure 6.11 Axial Load- Moment Design Strength Interaction Curve—30-in. Column

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:

$$V_n = cA_{cv} + \mu(A_{vf}f_y + P_c) \le (0.2f'_c A_{cv},\ 0.8A_{cv}) \tag{6-1}$$

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.

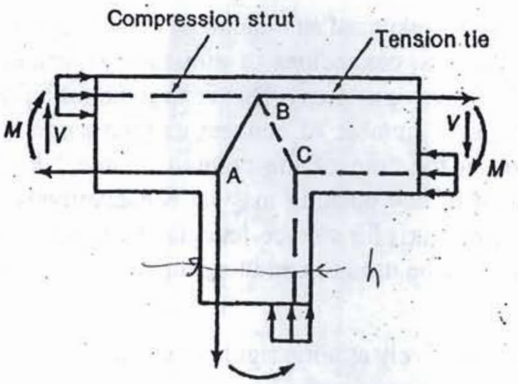


Figure 6.12 Beam-Column Joint Forces Based on Truss Model [6.26]

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:

$$V_n = 12\sqrt{f'_c}\, b_j h \tag{6-2}$$

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:

$$w = 0.076\beta f_s \sqrt[3]{d_c A} \tag{6-3}$$

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:

$$Z = f_s \sqrt[3]{d_c A} \tag{6-4}$$

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]:

$$s = \frac{540}{f_s} - 2.5c_c \leq 12\left(\frac{36}{f_s}\right) \tag{6-5}$$

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

$$= 24\sqrt{f'_c}\,h_{ef}^{1.5} \qquad h_{ef}\leq 11 \text{ in.} \qquad (6\text{-}8b)$$

$$= 16\sqrt{f'_c}\,h_{ef}^{5/3} \qquad h_{ef}>11 \text{ in.} \qquad (6\text{-}8c)$$

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 in²	d_b in	h_{ef} in	f'_c (ksi) concr	f'_c (ksi) grout	P_{yield} kips	P_{max} kips	u_{max} [A] psi	$u_{max}/\sqrt{f'_c}$	$u_{max}/\sqrt{f'_{cg}}$	$u_{dgx}/\sqrt{f'_c}$ [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.

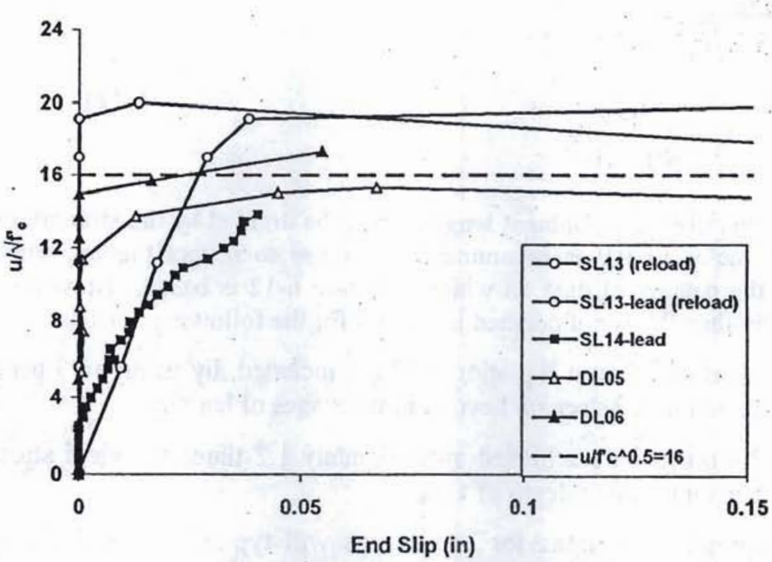


Figure 6.13 Normalized Bond Stress, $u/\sqrt{f'_c}$ vs. End Slip—Grout Pocket Straight-Bar Tests

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. $l_d=0.022 f_y d_b/\sqrt{f'_c}$

B. $P_{pred}=h_{ef}/l_d(A_s f_y)$, 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.

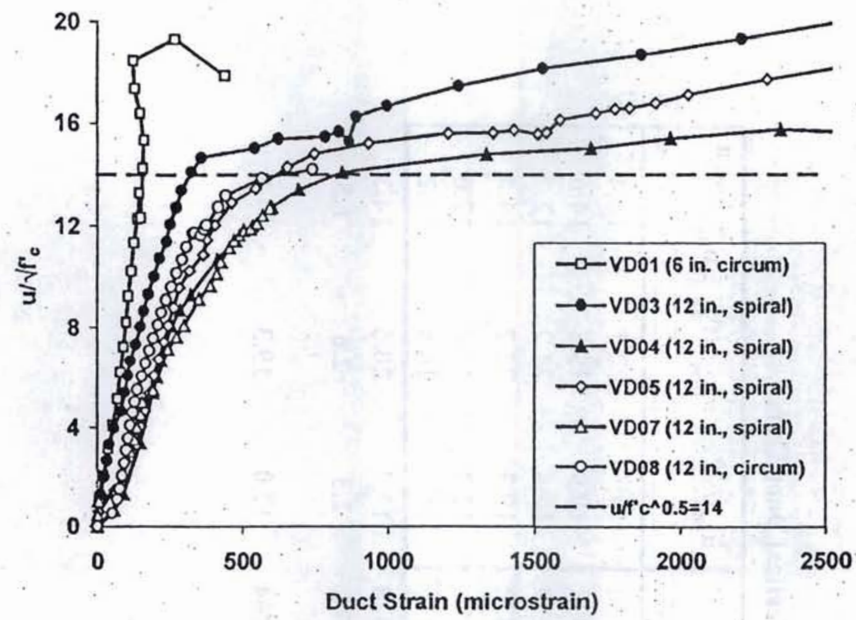


**Figure 6.14 Normalized Bond Stress, $u/\sqrt{f'_c}$ vs. Duct Strain—
Grouted Vertical Duct Straight-Bar Tests**

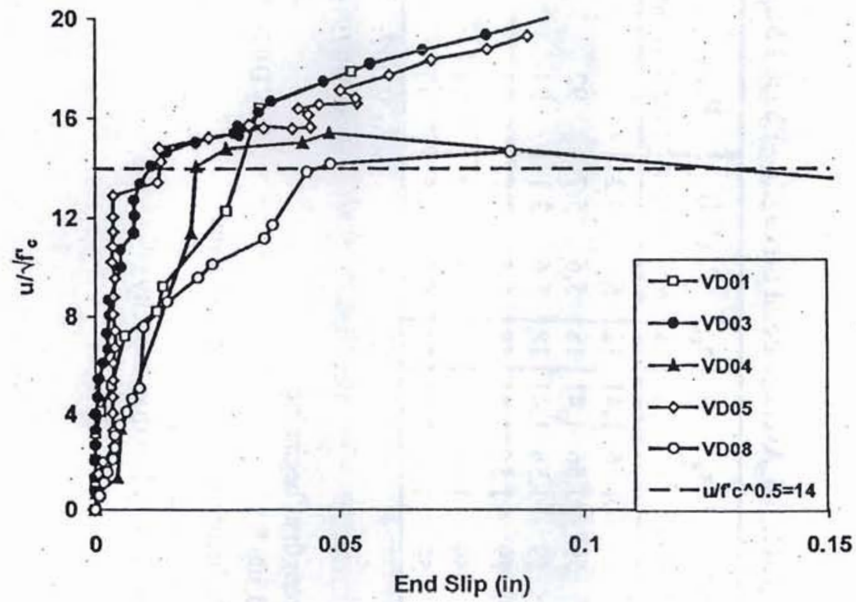


**Figure 6.15 Normalized Bond Stress, $u/\sqrt{f'_c}$ vs. End Slip—
Grouted Vertical Duct Straight-Bar Tests**

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

$$P = (1.25 f_y)\frac{\pi d_b^{\,2}}{4} = \frac{f_y \pi d_b^{\,2}}{3.20}$$

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
								ave	**1.7**

Footnotes

A. $l_d = 0.024 f_y d_b / \sqrt{f'_c}$

B. Ppred=$h_{ef}/l_d(A_s f_y)$, 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 ID	Bars no.-size	h_{ef} in	f'_c (ksi) concr	f'_c (ksi) grout	P_{max} kips/bar	A_N in^2	A_{NO} in^2	A_N/A_{NO}	c_{min} in	$1.5h_{ef}$ in	Ψ_E	Ψ_C	P_{CCD}[A] kips/bar	P_{max}/P_{CCD} (Ψ_C=1.0)	f'_{cl}/f'_c[C]	P_{max}/P_{CCD} (Ψ_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. $f_{cl}=[f_c(1-A_g/A_n)]+f_{cg}(A_g/A_n)$

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

$$= 40\sqrt{f'_c}\,h_{ef}^{1.5} \quad h_{ef}\leq 11 \text{ in.} \qquad (6\text{-}17b)$$

$$= 27\sqrt{f'_c}\,h_{ef}^{5/3} \quad h_{ef}>11 \text{ in.} \qquad (6\text{-}17c)$$

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.

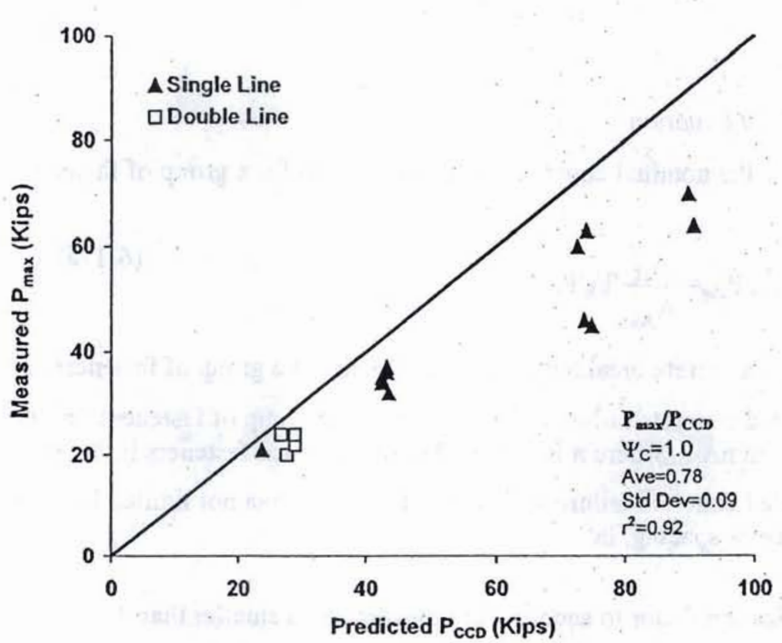


Figure 6.16 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests

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.

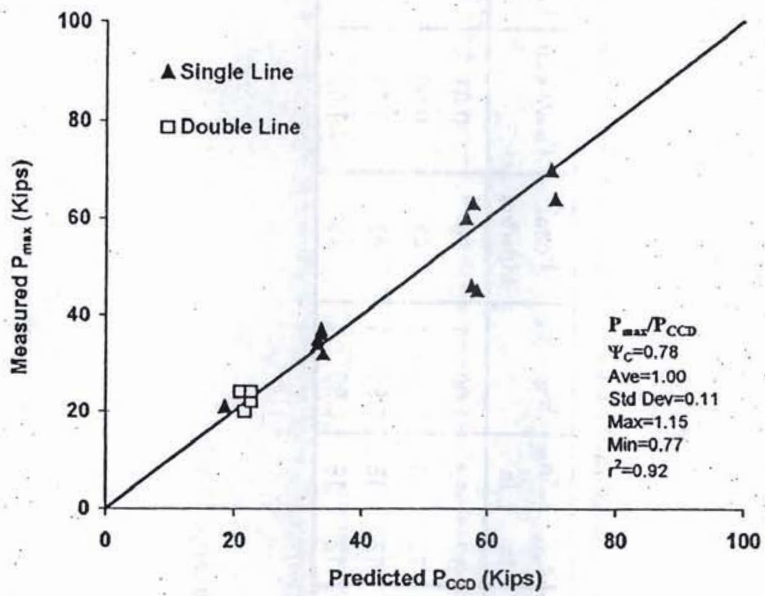


Figure 6.17 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests (Ψ=0.78)

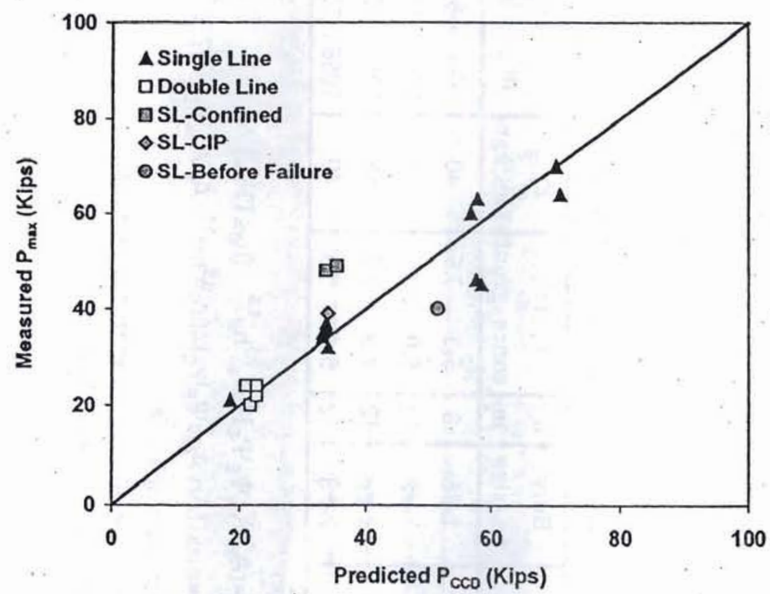


Figure 6.18 Comparison of Maximum Applied Load and CCD-Predicted Capacity—Grout Pocket Headed-Bar Tests (All)

Table 6.9 Comparison of Predicted and Actual Capacity—Additional Phase 1 Headed Bar Grout Pocket Tests

Test ID	Bars no.-size	h_{ef} in	f'_c (ksi) concr	f'_c (ksi) grout	P_{max} kips/bar	A_N in^2	A_{NO} in^2	A_N/A_{NO}	c_{min} in	$1.5h_{ef}$ in	Ψ_E	Ψ_C	P_{CCD}[A] kips/bar	P_{max}/P_{CCD}	f'_{c1}/f'_c[D]	P_{max}/P_{CCD} (Ψ_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:

$$P_{cbg} = \frac{A_N}{A_{No}} \Psi_E \Psi_C P_b \qquad \text{(6-18a)}$$

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

$$= 24\sqrt{f'_c}\,h_{ef}^{1.5} \qquad h_{ef} \leq 11 \text{ in.} \qquad \text{(6-18b)}$$

$$= 16\sqrt{f'_c}\,h_{ef}^{5/3} \qquad h_{ef} > 11 \text{ in.} \qquad \text{(6-18c)}$$

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 3 days 7 days 28 days	2500 4000 5000 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)	$3.0\text{-}5.0\times10^{6}$ psi	
Coefficient of thermal expansion (ASTM C-531)	$3.0\text{-}10.0\times10^{-6}$/deg F	
Constructability		
Flowability (CRD-C 621/ASTM C-939)	fluid consistency efflux time: 20-30 seconds	
Set Time (ASTM C-191) Initial Final	 3-5 hrs 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

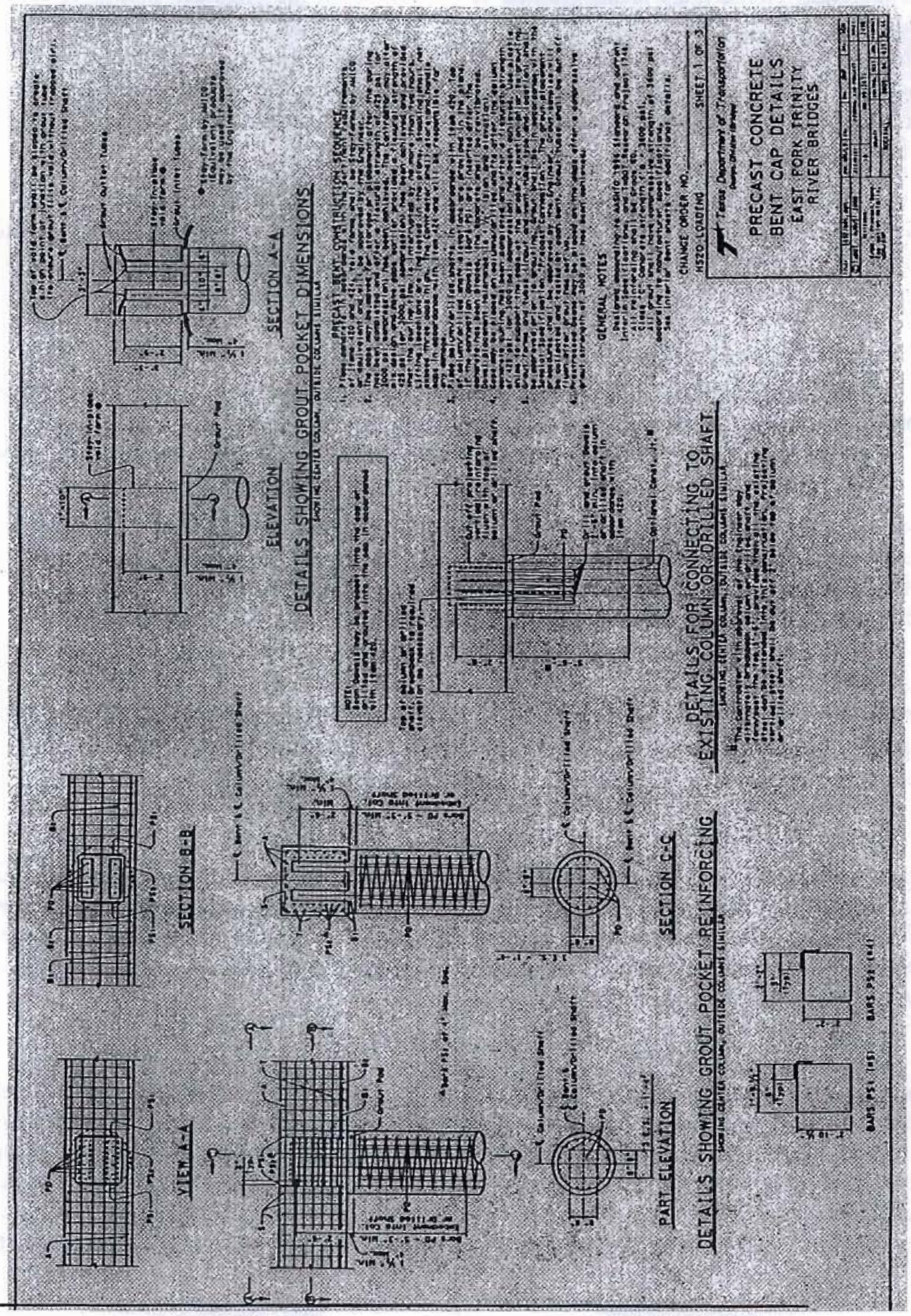
All exposed grout surfaces should be cured in accordance with manufacturer's recommendations. This will typically involve covering exposed grout with clean wet rags and maintaining moisture for a minimum of 6 hours, followed by the application of an approved membrane curing compound.

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.


Texas Department of Transportation
PRECAST CONCRETE
BENT CAP DETAILS
EAST FORK TRINITY
RIVER BRIDGES
VIEW A-A
SECTION B-B
PART ELEVATION
SECTION C-C
DETAILS SHOWING GROUT POCKET REINFORCING
ELEVATION
SECTION A-A
DETAILS SHOWING GROUT POCKET DIMENSIONS
DETAILS FOR CONNECTING TO
EXISTING COLUMN OR DRILLED SHAFT
PRECAST BENT CONSTRUCTION SEQUENCE
GENERAL NOTES