

Guidelines For Accelerated Bridge Construction Using Precast/Prestressed Concrete Elements Including Guideline Details



PCI Northeast Bridge
Technical Committee

Second Edition



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FORWARD

This guideline has been developed for the purposes of promoting a greater degree of uniformity among owners, engineers, and industry of the Northeast, with respect to planning, designing, fabricating, and constructing highway bridges with the FHWA's philosophy of accelerated bridge construction.

In response to needs determined by Northeast Transportation Agencies, and Prestressed Concrete Producers, the PCI Northeast Regional Bridge Technical Committee established a subcommittee comprised of a cross section of its members representing academia, transportation engineers, and producers to prepare this guide.

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Introduction

This guide is the second edition report developed by the PCI Northeast Bridge Technical Committee on the use of Precast/Prestressed Concrete Elements to accelerate the construction of bridge projects. The guide will assist designers in determining which means and methods would be appropriate for considering accelerated construction techniques. This guide will offer solutions from deck replacement to total construction of a bridge.

Some of the considerations for accelerated construction are:

- Improved work zone safety.
- Minimizing traffic disruption during bridge construction.
- Maintaining and/or improving construction quality.
- Reducing life-cycle costs and environmental impacts.

Precast elements produced off site can be quickly assembled, and can reduce design time, cost, minimize forming, minimize lane closure time, and/or possibly the need for a temporary bridge.

The use of precast elements such as abutments, pier caps, pier columns, and precast footings can effectively minimize construction time, traffic disruption, and the impact of construction activities on the environment.

This guide is organized in the customary order of bridge construction; essentially from the ground up. The manual starts with general information that applies to the whole structure. Following this, the reader will find specific information regarding the different precast elements used in accelerated bridge construction. Joints and grouting considerations may then be reviewed as the structures design becomes more defined. The final step then becomes construction. The reader will find recommendations regarding fabrication and inspection of each element used in the structure. Therefore, the reader will find the guide is divided into the following seven sections:

1. Application Overview
2. General Requirements
3. Precast Elements
4. Joints
5. Grouting
6. Seismic Considerations
7. Fabrication/Construction

Details have been developed for the various elements described in this guideline. These can be found in Appendix A. Each detail has been numbered for easier cross-referencing. For example: See Detail 3 on Sheet SUB 4.

This guide is not intended as a stand-alone document and does not supersede the AASHTO specifications

Section 1: Application Overview

1.1 When To Use Accelerated Construction

Accelerated construction techniques should be used where the benefits of accelerated construction have a positive effect on the construction costs and impacts of the project. In many cases accelerated construction techniques can reduce overall project costs. Bridge-specific costs on small, accelerated construction projects could be more than conventional construction for very rapid construction schedules (this is not necessarily the case with large scale projects). It is also anticipated that costs will come down as more accelerated projects are let. The savings in accelerated construction projects are found in other aspects of the project such as time, equipment use, and labor savings.

Decisions to use accelerated construction techniques should be made after considering the following issues:

- Temporary Roadways and Bridges
- Reductions in Environmental Impacts
- User Costs
- Political Pressures
- Long Detours

For additional guidance, refer to the Federal Highway Administration report titled “Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES), May 2006.” Several States have developed specific decision-making processes. Designers should check with the local DOT for guidance.

Accelerated construction should always be considered in cases where temporary bridges and roadways are anticipated. This is especially true where a reasonable detour is available. It may be desirable to close a roadway completely, build the bridge quickly, and live with a detour. In this case, the possible additional cost of the accelerated construction is far outweighed by the savings of not building a temporary roadway. Recent accelerated construction projects have shown that commuters and businesses prefer a significant short-term impact over a long-term moderate impact.

For bridges over water courses, impacts to the environment can be lessened by the elimination of a temporary bridge.

The cost of construction to highway users is significant. Savings to commuters are not typically reflected in construction budgets for highway projects; however there is a significant financial impact to the entire community due to travel delays. In many cases, the cost of accelerated construction techniques can be offset by reductions in user costs.

Often, the need for accelerated construction can be driven by political pressures. The impacts of construction on commuters and businesses in urban areas can be devastating. Accelerated construction can be used to limit the time frames for construction projects in these areas.

On some projects, the use of staging and temporary bridges is not feasible due to limited right of way and environmental issues. In these cases detours are the only option. Accelerated construction techniques should be considered if there are issues with traffic volumes on detours and access for emergency vehicles.

Though the intent of this manual is to provide information that applies to precast/prestressed elements used in bridge construction, using these elements in conjunction with non-precast elements is also encouraged. The designer may wish to use precast substructures with steel girders and precast deck panels, for example.

1.2 Rehabilitation Projects

Many bridge rehabilitation projects may benefit from accelerated construction methods. This guide focuses on precast elements that could replace the entire bridge; however, portions of existing bridges can also be constructed using these methods. The designer in these cases should balance the cost savings of not constructing new elements to the costs of rehabilitating existing elements. Costs should include both financial resources and time.

1.3 Examples Of Prefabricated Elements

Prefabricated elements in accelerated bridge construction are comprised of separately shipped pieces that are assembled in the field to form a larger structural element of the completed bridge. The **figures 1.3-1, 1.3-2, 1.3-3 and 1.3-4** show typical substructure units constructed with precast concrete elements. Superstructure elements are included in this manual, but not in great detail. More information on superstructure elements can be found in other PCI Northeast documents (www.pcine.org).

1.4 Architectural Treatments

An accelerated construction environment does not preclude the idea of having an attractive bridge. In fact the very opposite is the reality. With some careful planning, the resulting bridge can be built quickly and also be aesthetically pleasing.

In most cases, cost will not be a limiting factor. Precast elements allow for architectural enhancements at a relatively lower cost than cast-in-place concrete. All treatments

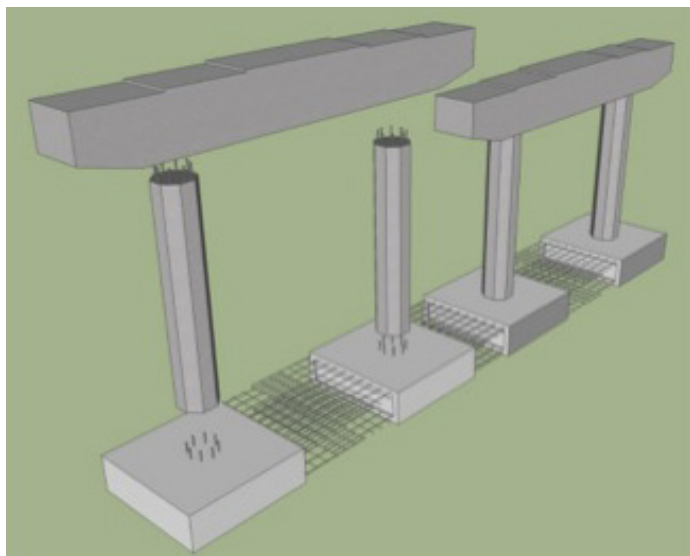


Figure 1.3-1 Pier Bent Elements (Ref: PCINE Sheet SUB-2&3)

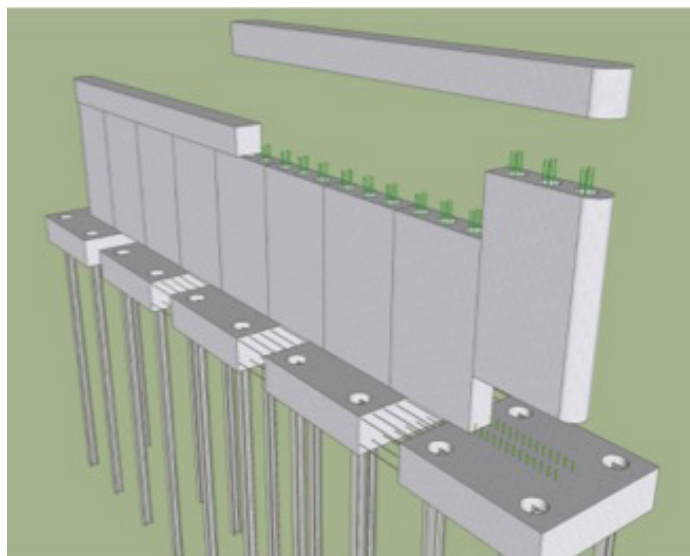


Figure 1.3-2 Wall Pier Elements (Ref: PCINE Sheet SUB-4)

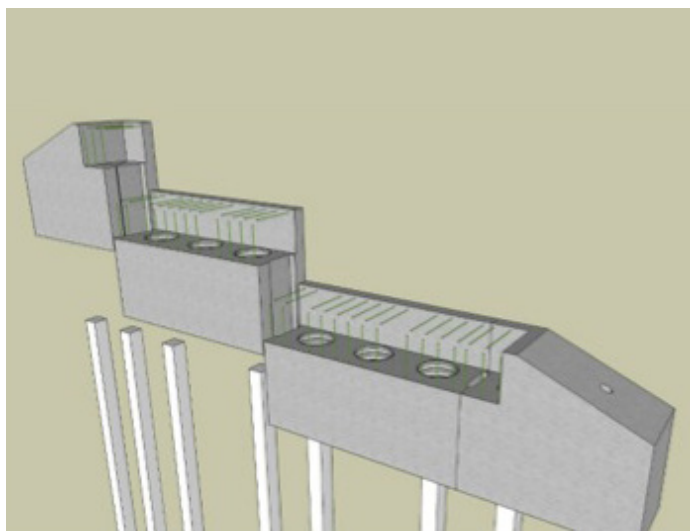


Figure 1.3-3 Integral Abutment Elements (Ref: PCINE Sheet SUB-8)

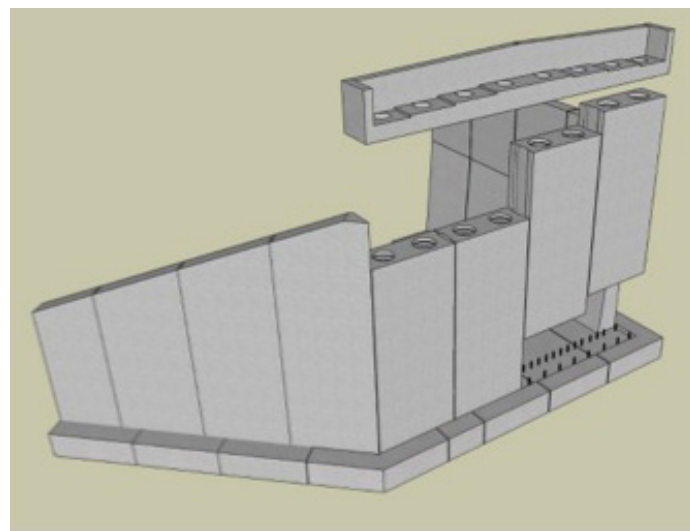


Figure 1.3-4 Cantilever Abutment Elements (Ref: PCINE Sheet SUB-9)

are made at the precast plant where repetitive use of standardized forms lowers the costs to individual projects. Precast plants are well suited for applying aggregate surfaces through means of blasting or the use of retardants.

The *PCI Bridge Design Manual* and the Minnesota Department of Transportation's *Aesthetic Guidelines for Bridge Design* offer guidance on this topic. These guidelines may be used to proportion elements to fit together to meet the function of the structure, as well as to enhance aesthetics.

Section 2: General Requirements

Guidelines

2.1 Partial Replacement Projects

If the existing substructure is to be reused, complete dimensions and elevations should be obtained to ensure compatibility with the new precast elements.

2.2 Design

Providing a safe design to meet the site requirements is paramount in all bridge replacement projects. The engineer must focus on ease of fabrication, repetition, and ease of assembly to create a cost-effective, precast concrete solution.

In general, the design of precast substructures involves emulation of traditional cast-in-place concrete structures with discrete precast elements. The connections between elements are designed to emulate traditional construction joints.

Designers may take advantage of post-tensioning technologies to facilitate construction of complex structures.

The design and detailing of beams and girders is generally not affected by accelerated construction techniques.

2.3 Geometric Configurations

2.3.1 Bridge Layout

Non-skewed designs are preferred.

2.3.2 Element Sizes and Shapes

The designer should detail elements sizes to promote repetition of forming with consideration given to transportation, fabrication, and construction.

Battered elements should be avoided.

2.4 Tolerances

Designers should specify and account for tolerances in layout of elements. Section 7.5 contains more information on tolerances for common elements.

Commentary

There is adjustability in precast elements; however the tolerances at interfaces are limited. The field survey is recommended.

Designers should refer to the ACI 550.1R-01, *Emulating Cast-in-Place Detailing in Precast Concrete Structures* for specifications on emulation design.

It is preferable to have angles between abutment and wingwalls that are in-line or 90 degrees, although odd angles can be accommodated.

Footing widths may be detailed such that there are common dimensions on each bridge project. For instance, on a particular bridge, all footings for wingwalls that are of approximately equal height could be kept identical (dimensions and reinforcing). The economies of repetition may outweigh the perceived benefits of individually sized elements.

Batters on abutment and wing stems should be eliminated and the overall thickness of the stems should be minimized to reduce the overall weight of the element. Wall type elements typically are cast horizontally as slabs.

All precast concrete products are constructed within a specified tolerance. The PCI Northeast Bridge Technical Committee has established tolerances for common bridge elements, which are based on the *PCI Tolerance Manual MNL 135*.

Erect and lay out elements based on common working lines.

Nominal joint widths should be set based on the specified tolerances.

2.5 Shipping and Handling

The size of precast elements should be determined with consideration for shipping restrictions, equipment availability, and site constraints.

In special cases, very large pieces can be detailed; however the shipping, handling, and installation costs should be considered.

The designer should consider each state's requirement for allowable shipping widths. The width of the element shall include all protrusions.

Precast elements shall be checked for stresses induced during handling and shipping. The design for handling is the responsibility of the fabricator. The *PCI Design Handbook*, as well as lifting device manufacturer's recommendations, should be specified as a reference for handling calculations. This requirement shall be included in the contract documents.

2.5.1 Lifting Devices

The design and detailing of lifting devices is the responsibility of the fabricator. Lifting devices should be placed to avoid being visible once the precast element is placed. Lifting devices that are located in areas that will be visible or exposed to the elements should be detailed with recessed pockets that can be patched after installation. The patching material should approximate the appearance of the surrounding concrete and provide corrosion protection. **See Section 7.1.**

Base the layout of elements on dimensions measured from a common working line as opposed to nominal center-to-center spacing. The use of center-to-center spacing can result in loss of overall structure geometry due to cumulative erection tolerances (see the *Typical Guideline Details for Precast Concrete Structures*).

At a minimum, the joint width should account for the fabrication and erection tolerances of the elements. The PCI Northeast Bridge Technical Committee has established tolerances for common bridge joints based on this approach. **See Sheets SUB 5, 6, and 10.**

The weight of precast substructure elements weighing on the order of 30 tons should be anticipated.

It is possible to ship pieces in excess of 30 tons; however the equipment required and limitation of local bridge capacities may restrict this. Off-loading of pieces can also be problematic. Larger pieces may be feasible if the pieces can be fabricated in close proximity to the bridge and shipped a short distance.

In general, elements should have a maximum width of 12 feet to avoid cost premiums typically associated with shipping of large elements over the road. Elements with widths in excess of 12 feet typically require special trucking permits, which can be supplied at a premium.

Reference: Latest edition of the *PCI Design Handbook*

The designer should specify the level of corrosion protection for lifting devices.

Section 3: Precast Elements

Guidelines

3.1 Foundation Elements

Foundation elements include piling, sheet piling, pile caps, and footings.

3.1.1 Piling

(Ref: *PCINE Sheets SUB-7 & 8*)

The designer may choose to use precast prestressed concrete piles as an alternative to steel 'H'-piles. Consult the project geotechnical report for specific limitations regarding the project site before selecting the pile type and size.

3.1.2 Footings

(Ref: *PCINE Sheet SUB-7*)

The transfer of footing loads to the underlying soils should be made via a filled gap below the footings (grout or flowable fill).

The most common standard finish on the underside of footings is a form finish. However, if greater coefficients of friction are required, several options are available to achieve the desired results. Costs of formwork and handling should be considered in determining the best approach.

3.1.2.1 Construction on Bedrock

A more extensive soils boring program should precede construction of precast footings so that the degree of variation of top of rock elevations can be assessed prior to construction.

The uneven nature of construction of footings on bedrock may require preparation of the site prior to installation of precast footings. Over-blasting of rock by approximately 12" to provide room to prepare for a relatively level work area is recommended. This will facilitate the installation of grout or flowable fill under the footings. See **Section 3.1.2.5**.

Once the area is made roughly level, there are two recommended methods for preparing the area for installation of precast footings. The first is to pour a low-strength concrete sub-footing that provides room for grouting. The second method is to provide small level concrete surfaces under the proposed leveling devices. See **Section 3.1.2.4**.

Commentary

Practice has shown that a minimum of 14-inch prestressed pile sections has been successfully used in severe driving conditions. For more information regarding precast/prestressed concrete piles, refer to the *PCI Bridge Design Manual* BM-20-04 chapter 20.

It is unreasonable to assume that proper interface can be achieved between compacted soil and a precast element. The unevenness of compacted soil combined with the tolerances of precast will lead to point of localized support. An effective means of providing this support is a grout-filled gap.

Formliners and keyways are the most economical production practices. Sandblasting/ exposed aggregate finishes are also available; however, handling requirements may require rolling of panels. A rake finish with $\frac{1}{4}$ " amplitude is also an option but would require specialty forming and handling needs.

As with any construction on bedrock, large variations in rock elevations can affect the layout and design of precast substructure elements. It may be desirable to step footings where rock variations are significant. The contractor will also need this information to plan the work. Unknowns in rock elevations are always difficult to address. It is essential that most of this be addressed prior to construction on an accelerated project. The owner should balance the need for more borings with cost constraints.

The reason for over-blasting is to ensure that the removal of rock will be a one-time process, and the amount of post-blast clean-up removal will be kept to a minimum.

3.1.2.2 Construction on Soil

(Ref: PCINE Sheet SUB-7)

Prior to construction on soil, the area must be excavated and prepared as in normal cast-in-place construction.

Once the area is prepared, there are two recommended methods for preparing the area for installation of precast footings. The first is to pour a low-strength concrete sub-footing to a level that is just below the proposed bottom of footing elevation as shown in **Figure 3.1.2.2-1**.

The second method is to provide small level areas under the proposed leveling devices. See **Section 3.1.2.4**. Temporary load distribution plates will be required under the leveling devices when a sub-footing is not used in order to spread the loads to the soil. This method is more cost effective, therefore it should be considered for most situations.

3.1.2.3 Construction on Piles

(Ref: PCINE Sheets SUB-7 & 8)

Construction on piles will, in general, follow the guidelines for construction on soil. A concrete sub-footing may be used, or the footing can be temporarily supported on load distribution plates on soil.

Provisions should be made in the footing design for concreting of the areas around the pile tops. Concrete is placed in the voids after setting of the pile cap footing or integral abutment cap.

Provide clearance around each pile to account for driving tolerances. **See Notes 5 on Sheet SUB-1.**

Designers should consider the use of cast-in-place concrete footings for foundations with closely spaced piles and large foundations that lead to very heavy footing elements.

The concrete sub-footing need not be high strength. The typical range of footing pressures are magnitudes less than the strength of the sub-footing concrete. The sub-footing concrete need not be formed. In most cases, the concrete can be cast against the footing excavation limits. Experience has shown that a low-strength concrete sub-footing does not slow construction and provides a very good work platform for installation of precast elements.



Figure 3.1.2.2-1 Placing footing segment on a sub-footing.

Typical state standards for pile installation tolerance are +6". This tolerance is normally too large for precast footings, since the voids in the footings need to be sized to accommodate the pile installation tolerance. In many cases, it is possible to install piles to a smaller tolerance through the use of driving frames and templates. Tolerances of +3" have been attained without significant difficulty. Designers should consult with the project geotechnical engineer and the state agency prior to reducing the pile installation tolerance.

Closely spaced pile voids create difficulties with the spacing of reinforcing bars. In this case, the precast sub-structure stem can be set on temporary support struts, and the reinforcing bars can be projected into the footing pour, eliminating the need for connection hardware at the interface between the two elements. **See Detail 6 on Sheet SUB-7.**

Construction of foundations in deep water can be facilitated through the use of perched cast-in-place footings above the bottom of the body of water. Precast concrete elements can be used to form the footing and provide a dry work area for the footing installation.

3.1.2.4 Leveling Devices

(Ref: *PCINE Sheet SUB-7*)

Leveling devices are critical in maintaining proper vertical grade control on precast concrete substructures. Cast-in embedded leveling devices should be used to allow for adjustment of the footing grade and elevation during installation.

A minimum of four leveling devices should be specified for each spread footing element. Each device should be designed to support half the self-weight of the footing element.

The element should be leveled prior to release of the piece from the crane. A thorough greasing of the leveling device is recommended. See Detail 4 on Sheet SUB-7.

Once the installation of the element is complete, the leveling bolt may be left in place or backed out and the blockout filled with grout.

3.1.2.5 Grouting Under Footings

(Ref: *PCINE Sheet SUB-7*)

The purpose of grouting under spread footings is to distribute the foundation pressures from the precast footing to the underlying soil or rock. A gap that is grouted is required to achieve this. Exact grouting methods can be left up to the discretion of the general contractor. The plans and specifications should give certain guidelines on grouting procedures. See Section 5 for more information on grouting.

Precast cofferdams are specialized structures that are specific to each project, which requires specialized design and detailing. **Figure 3.1.2.3-1** shows a precast cofferdam that was used on the Providence River Bridge in Providence, R.I.



Figure 3.1.2.3-1 Precast Concrete Cofferdam.

Experience has shown that these leveling devices provide fast and easy grade adjustment at a minimal cost. The use of leveling shim packs is discouraged since there is no way to adjust the grades without removing the element.

During installation, there is a tendency for the piece to rock on the diagonal corner supports, therefore each device should be designed to support half the weight of the element.

The effort to adjust the leveling devices is greatly reduced if the element is partially supported by the crane, or if it is greased.

There are several methods that have been successfully used. The contractor should be allowed to use a method that best suits the experience of the workers and the available equipment.

The strength of the grout is secondary to its ability to properly fill the gap under the footing. Flowable fill is another option for a fill material that can be used.

The fill material should be placed in the void through ports cast in the footing. Attempting to flow the grout from one side to another is not recommended unless the footing is relatively narrow.

3.2 Substructure Elements

Substructure elements include wall segments, columns used in piers, approach slabs, and pier caps.

3.2.1 Retaining Wall Elements

There are several wall options available to designers for accelerated construction projects. Many States maintain approved proprietary precast concrete retaining wall systems. Another option is to use a precast concrete cantilever wall. The following options should be evaluated for each wall:

3.2.1.1 Cantilever Walls

(Ref: PCINE Sheet SUB-10)

Cantilever walls are concrete walls consisting of a concrete footing combined with a concrete vertical wall element. The footing engages the backfill material to resist the overturning and sliding forces acting on the wall.

Cantilever retaining walls can be detailed using the techniques outlined in this guideline. The wall stems and footings can be made with precast concrete elements.

Cantilever wall footings can be placed on sub-soil (spread footing) or on piles.

See Detail 8 on Sheet SUB-10.

In most cases, the foundation bearing pressures are significantly less than the capacity of most grouts. A more cost-effective material for this situation is flowable fill (also referred to as controlled density fill). Flowable fill typically has strength that is sufficient to support most spread footings. It can be placed in small voids and has sufficient fluidity to spread under the footing between the fill ports.

Placement may be accomplished by pumping or gravity feed through grout ports. The ports should be arranged so that the grouting operation progresses in a single general direction to avoid air pockets.

Designers should refer to each state's specifications for a listing of the approved proprietary walls and for the proper methods for specifying them.

Often this type of wall will use the least amount of width (normal to wall face) when compared to other retaining wall systems.

3.2.1.2 Soldier Pile Walls

(Ref: *PCINE Sheet SUB-10*)

Soldier pile walls consist of vertical pile elements that are installed at regular intervals combined with prefabricated panels that are placed between the piles to retain the earth.

Soldier pile retaining walls can be detailed using the techniques outlined in this guideline for integral abutments. The wall stems can be made with precast concrete elements.

For tall walls, or walls that are built over shallow bedrock, the resistance of the vertical piles can be increased through the use of soil or rock anchors placed at regular intervals.

See Detail 9 on Sheet SUB-10.

3.2.1.3 Mechanically Stabilized Earth (MSE)

Mechanically stabilized earth walls are designed based on the technique of engaging the soil mass behind the wall face through the use of layers of reinforcing strips or grids. Once engaged, the soil mass provides a gravity unit that can resist overturning and sliding. The reinforcing strips can be made of corrosion protected steel, or polymers.

A precast concrete facing element is typically used to retain the soil near the face of the wall.

3.2.1.4 Concrete Modular Block Gravity Walls

Retaining walls can be constructed with precast concrete modular elements that are filled with compacted soil or crushed stone.

3.2.1.5 Sheet Piling Wall

Sheet piling retaining walls can be constructed using precast prestressed interlocking sheet elements that are placed in line to form a cantilever wall. The top of the wall is typically capped with cast-in-place concrete.

Many contractors use temporary soldier pile walls for excavation support. In these cases, timber panels are typically used as lagging between the piles. Permanent soldier pile walls are made with precast concrete panels with the pile embedded in the precast element for long-term durability.

Using MSE walls is an ideal solution for accelerated construction. The wall facing, reinforcing strips, and backfill can be constructed concurrently, which makes this wall type ideal for fill situations.

A precast concrete modular block gravity wall is another ideal solution for accelerated construction. The blocks interlock using keys cast into them. The dead weight of the blocking system along with the interlocking keys eliminates the need for mechanical connections between precast units.

Modular block walls are typically proprietary items. Refer to state-approved wall suppliers.

The sheets support the soil through cantilever action. For tall walls, soil anchors can be used to help resist the lateral soil forces.

These walls have been used in favorable soil conditions (sandy soils). Hydro-jetting is often used to facilitate sheet installation.

Details for sheet piling walls can be found at the Florida DOT website.

3.2.2 Columns

3.2.2.1 Column Shapes

(Ref: PCINE Sheet SUB-5)

Rectangular, hexagonal, or octagonal columns are a preferred choice to facilitate fabrication.

Round columns are possible, but more difficult to fabricate. These will likely have to be poured vertically which may prove to be difficult in a precast plant. This will likely result in higher element prices.

Columns with at least one flat face can be poured on their sides. Several can be poured at the same time – side by side. This can enhance the efficiency and therefore reduce the cost of the element.

Hexagonal and octagonal columns can be detailed with spiral reinforcement that mimics the behavior of round columns.

3.2.3 Girder Support Elements

Precast elements can be used to distribute girder loads to foundations. The most common elements are as follows:

3.2.3.1 Pier Caps

(Ref: PCINE Sheets SUB 2, 3 & 4)

A pier cap is a beam that spans the columns it is being set upon. The cap is connected to the columns by grouted mechanical splices.

Pier caps can be supported by single columns, or two or more columns. Assembly of a pier cap is shown in **Figure 3.2.3.1-1**. General pier construction is shown in **Figure 3.2.3.1-2**.

In seismic zones 3 and 4, place grouted mechanical splices in the pier cap and footing if possible in order to avoid issues with splices in the column plastic hinge zone. See **Detail 5 on Sheet SUB-5** for placement of splices in footings.

PCI Northeast details are based on the use of grouted mechanical splices. Pier caps can also be connected using post-tensioning systems.

It is recommended that individual pier cap elements be connected to three or less columns. This is due to the difficulties of connecting many elements together in the field. If a bridge has more than three columns, two options are recommended:

- Use multiple pier cap elements and leave an expansion joint between the caps.
- Connect the caps after installation using a closure pour. Design the cap to support the dead load of the superstructure so that the casting and curing of the closure pour will not delay construction.



Figure 3.2.3.1-1 Assembly of a Pier Cap.



Figure 3.2.3.1-2 Pier Construction

3.2.3.2 Integral Abutment Pile Caps

(Ref: **PCINE Sheet SUB-8**)

Pile caps are typically used in integral abutment bridges. There are two types of integral abutments: fully integral and semi-integral. Fully integral abutments provide a full moment connection to the superstructure while semi-integral abutments have a pinned connection to the superstructure allowing the superstructure to rotate. See individual state agency guidelines and details for the each state's preferred approach.

Corrugated metal pipe is an effective method of creating the void to accept the pile and making the connection between the pile and abutment cap. **See Section 4.4.3** for more information. The designer should specify the acceptable pipe corrugations.

In larger precast sections, corrugated pipe can also be used to create additional voids to reduce the weight of individual sections. An example of a pile cap in an integral abutment structure can be seen in **Detail 2 on Sheet SUB-8**.

The tolerance for this type of construction is the same for footings. **See Notes 5 on Sheet SUB-1**.

Wide abutments may require the caps to be fabricated in sections. Options for connecting adjacent sections include:

- Match casting and post tensioning
- Vertical shear key
- Small closure pours

The key feature of this method is the corrugations in the pipe. These corrugations are necessary to transfer the forces between the concrete cast within the pipe and the surrounding precast concrete. It is important to specify a pipe that has continuous and uniform corrugations along the entire length of the pipe so that the full concrete section can be engaged for shear transfer.

The following table includes typical corrugations sizes for corrugated metal pipes:

Inside Diameter Range	Corrugation Pattern
4" to 18"	1.5" x 0.25"
12" to 84"	2.66" x 0.5"
36" to 144"	3" x 1" and 5" x 1"

Corrugated steel pipes should be galvanized for corrosion protection.

Some pipe manufacturers produce pipes with low flow friction walls that are designed to convey water more efficiently. Corrugated plastic pipe are also available. These types of pipes should only be used for non-structural voids.

The PCI Northeast details show the vertical shear key connection, which is the preferred connection, due to its simplicity and cost effectiveness.

3.2.3.3 Wall Caps

(Ref: PCINE Sheets SUB 4 & 9)

Precast caps should be used on top of wall stem elements to simplify the detailing of the substructure.

3.2.4 Approach Slabs

(Ref: PCINE Sheets APP-1 through 5)

Approach slabs are designed to span between the bridge abutment and the approach fill. Details for approach slabs vary by agency. Precast approach slabs can be used and detailed to meet the specific requirements of each agency.

Approach slabs need not be supported on the substrate over their entire length. The slab only needs to be supported at the ends. The slab is normally supported by the abutment on one end and soil, or a sleeper slab at the far end. The area between the two supports need not be in contact with the ground.

Beam seats typically require complicated details in order to provide variable beam seat elevations. The use of a precast cap allows for the complex detailing to be limited to fewer elements as opposed to each wall stem element. Precast abutment caps can also be detailed to include integral backwalls and cheekwalls.

The connection of the wall stems to the caps can be done through the use of reinforcing bar dowels placed within a corrugated metal pipe void in each element.

(See Detail 4 on Sheet SUB-6.)

Precast wall caps can be used on projects where the substructures are to be rehabilitated. The existing substructures can be removed to an elevation that is approximately 2 feet below the new beam seat. The new wall cap can then be connected through the use of grouted dowels.

The purpose of approach slabs is to span over potential settlement of the soil directly behind the abutment. Most states set approach slabs at the surface of the roadway. Some states set the approach slabs below grade and fill on top of the slab with subbase and pavement. Buried approach slabs perform the same purpose as surface approach slabs, except that the slabs are better protected from deicing salts.

Various details have been developed by the PCI Northeast Bridge Technical Committee that depict various options that correspond to different state standards. The committee has also included details of precast barriers that are integral with the approach slab. Designers should select details that are consistent with the agency standards.

There are proprietary approach slab systems in the market that can be used.

The use of grouts under approach slabs is not required since the soil will most likely settle over time, leading to gaps. This is accounted for in the design of the approach slab. Flowable fill, which is less expensive than grout, can be used to seat the approach slab on the substrate at the ends, or if desired, along the entire approach slab.

3.3 Superstructure Elements

3.3.1 Girders and Beams

Girders or beams shall be designed and detailed according to conventional methodology. Refer to the *PCI Bridge Design Manual*, the *AASHTO LRFD Bridge Design Specifications*, and specific state agency standards.

3.3.1.1 Bulb Tee Girders (NEBT) & (PCEF)



Figure 3.3.1.1-1 NEBT.

Bulb tee girders are precast/prestressed girders that are designed to be more efficient than traditional AASHTO I girders. It is a stringer bridge system with a concrete deck that is either cast-in-place or precast. It was developed for bridge spans 80 feet and longer. The design of the girders could be simple span up to 160 feet or be spliced

to achieve spans up to 200 feet. Standard girders are available in depths of 39.4 inches (1000 mm) to 86.6 inches (2200 mm). Specialty girders can be fabricated beyond this limit. A typical NEBT Girder is shown in **Figure 3.3.1.1-1**.

3.3.1.2 Northeast Deck Bulb Tee

A deck bulb tee is a bulb tee girder with an integral deck that is cast monolithically and prestressed with the girder.



Figure 3.3.1.2-1 Deck Bulb Tee Construction

Load transfer between adjacent units is accomplished using specially designed connections along with a reinforced concrete closure pour. After the shear keys are grouted, the beams typically receive an asphalt or concrete overlay.

The PCI Northeast Bridge Technical Committee developed the Northeast Bulb Tee (NEBT) in 1995. This girder was developed as a metric section, which is the reason for the uneven girder depths. The girders are occasionally referred to with their metric depth designations.

Deeper girders can be considered on a case-by-case basis. Designers should contact fabricators to determine feasibility and shipping implications.

The PCEF (Prestressed Concrete Committee for Economic Fabrication) girder was developed by the mid-Atlantic states and is similar to the NEBT. The two are commonly interchanged with owner approval in the Northeast.

The PCI Northeast Bridge Technical Committee is in the process of developing standard sizes and details for deck bulb tee girders based on the Northeast Bulb Tee Girder. The deck bulb tee's top flange is wider and thicker than the equivalent bulb tee. Deck concrete will not be cast in the field for this bridge type. A wearing course is recommended but not mandatory.

NYSDOT and MassDOT have built projects with deck bulb tee girders already. **Figure 3.3.1.2-1** shows a deck bulb tee bridge under construction in Massachusetts.

The committee is investigating alternate flange edge connection details including the use of Ultra-High Performance Concrete. It is anticipated that multiple details will be allowed for this connection.

3.3.1.3 Northeast Extreme Tee (NEXT) Beam

The Northeast Extreme Tee (NEXT) beam is a precast/prestressed, double-stemmed beam element. The beam is designed to either support field cast concrete or to function as the deck of the bridge. The beam span range is approximately 40 to 90 feet and it comes in depths from 24" to 40".

NEXT Beams are available in two types. The first type is the NEXT F beam (F stands for "form"). The top flange of the beam is used to support a field cast reinforced concrete deck.

The second type is the NEXT D beam (D stands for "deck"). The NEXT D beam has an 8" flange and requires a small closure pour cast in the field between the beams.

3.3.1.4 Box Beam, Deck Beams, and Slabs

Adjacent Box Beam systems consist of rectangular precast/prestressed concrete box sections that are placed side by side to form the entire superstructure. Beams can span from 20 to 120 feet and the depth and beam type will vary based upon span.

The system uses a grouted shear key and lateral post-tensioning to connect the beams. Some states have chosen to add a 5" to 6" thick reinforced concrete overpour to improve the performance of the system.

For more information on the design and construction of deck bulb tee beams refer to NCHRP 12-69 Final Project Report, "Guidelines For Design And Construction Of Decked Precast, Prestressed Concrete Girder Bridges."

PCINE Bridge Technical Committee developed this beam in 2008. The beams were developed for the medium-span bridge market. The section is simple and rugged. It is economical to construct with all strand straight and the stable shape of a double tee. **Figure 3.3.1.3-1** shows a NEXT Beam after fabrication.

This beam offers the advantage of reduced forming and subsequent form stripping in the field. The beam also provides a safe work platform immediately after beam placement.

The NEXT D beam allows a contractor to open a road to traffic within days of beam placement.



Figure 3.3.1.3-1 NEXT Beam

The untopped version allows a contractor to open a road to traffic within days of beam placement.

Deck slabs are the thinnest section and have no voids. Deck beams are thicker and have multiple round voids. Box beams are the thickest section and have rectangular voids. **Figure 3.3.1.4-1** shows a deck slab after fabrication.



Figure 3.3.1.4-1 Deck Slab



Figure 3.3.2-1 Placing Deck Panels

3.3.2 Full-Depth Deck Panels

Prefabricated decks offer advantages for deck construction since elements can be prefabricated offsite and assembled in place. Other advantages include removing the deck placement, deck forming, and concrete curing from the critical path of bridge construction schedules, cost savings, and increased quality due to controlled factory conditions. **Figure 3.3.2-1** shows construction of a full-depth deck panel project in New Hampshire.

Re-decking with prefabricated modular deck panels is a viable method of deck replacement that minimizes traffic disruption. More importantly, this construction method allows opening part of the bridge under construction to traffic. In addition, nighttime re-decking with prefabricated concrete modular panels, although slightly more costly than daytime re-decking, can further minimize interruption of traffic. Also, the existing composite concrete deck could be replaced in stages. In each stage, a portion of the transverse section is removed and replaced along the full length of the bridge, while other lanes are maintained open for traffic.

General information on full-depth deck panels is presented here. For more information, refer to “*Full Depth Deck Panels Guidelines for Accelerated Bridge Deck Replacement or Construction*,” PCINER-11-FDDP. This document is available at the PCI Northeast website (www.pcine.org).

PCI SOA-01-1911, “*State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels*,” is also available at www.pci.org.

3.3.3 Partial-Depth Deck Panels

In situations where a cast-in-place deck will be necessary, precast partial-depth deck concrete panels may be used to save time during construction. These panels do not require the extensive shoring and carpentry that conventional wood forms require, nor do they need to be removed once the deck has cured.

3.4 Proprietary Bridge Systems

The use of proprietary bridge systems can be considered with approval from the bridge agency.

3.5 Bridge Railing And Parapets

The designer should use parapet and/or rail systems that meet the owner agency's requirements. The FHWA and the AASHTO LRFD Bridge Design Specifications require that railing and parapet systems be crash tested.

NEXT beams offer opportunities to use standard crash-tested parapets built with precast concrete. The connection of the precast parapet can be made by casting the deck against the side of the precast parapet with projecting reinforcing. **Figure 3.5-1** is a detail of this approach. A similar connection can be used if the bridge is built with a sidewalk.

General information on partial depth deck panels is presented here. For more information, refer to "*Precast Deck Panel Guidelines*," PCINER-01-PDPG. This document is available at the PCI Northeast website (www.pcine.org).

The PCI Northeast Bridge Technical Committee has developed typical details for NEXT beam bridges. The details are available at the PCI Northeast website (www.pcine.org).

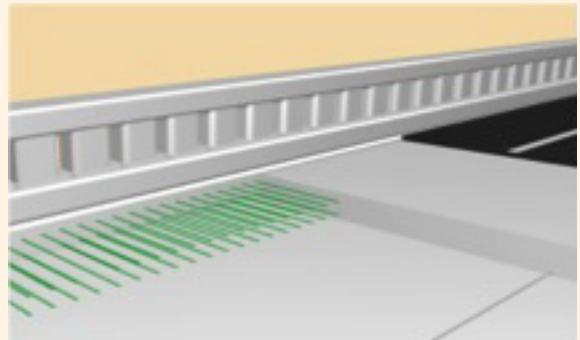


Figure 3.5-1 Precast Parapet Connection with NEXT F Beams

Section 4: Joints

Guidelines

4.1 General

Joints fall under two categories. The first are structural connections that transmit moment, axial, or shear forces between elements. The second are non-structural connections that may be used for thermal movements or to separate discrete portions of the structure (e.g., abutment to wingwall joint).

4.2 Layout of Joints

In general, the designer should show proposed layout plans of all joints that form connections in the structure. This layout plan will be used as a guide to determine sizes of elements and general construction sequencing.

The designer should include contract provisions that allow different joint configurations within contract defined boundary conditions.

4.3 Joint Width and Tolerance

(Ref: PCINE Sheets SUB-6 & 10)

The width of joints is set based on several factors:

1. The tolerance of the two joined elements,
2. The erection tolerance of the two joined elements.
3. The minimum joint width required to insert the filler material.

Knowing these values, the design engineer can specify a joint width and a joint width tolerance. The following sections contain recommended equations for use in determining joint widths.

Commentary

Full-height elements with vertical joints are typically preferred over elements that are “stacked” with horizontal joints. However, horizontal joints may be incorporated in a design if the weight or size of the pieces is excessive.

Locations and configuration of joints should be the contractor’s option based on boundary conditions set by the designer.

Examples of boundary conditions are as follows:

- The designer may specify that a vertical joint be placed away from bearing locations.
- The designer may specify a minimum width of elements.
- Stage construction joint locations may need to be specific.

4.3.1 Vertical Joints

(Ref: **PCINE Sheet SUB-10**)

The width of vertical joints needs to account for the following:

1. The possibility that one or both of the adjacent elements will be fabricated wider or longer than detailed, but within the specified element tolerance.
2. The possibility that one or both of the adjacent elements will be erected closer together than detailed but within the specified erection tolerance.
3. The minimum width of the joint must accommodate the joint filler material after all tolerances are accounted for.

Based on this, the minimum required joint width specified on the plans would be the absolute minimum tolerable joint width plus the half the width tolerance of the two adjacent elements plus the erection tolerance of the two adjoining elements.

The equation for the specified vertical joint width would be:

$$W_j = W_{\min} + 1/2 * (w_{tl} + w_{tr}) + (e_{htl} + e_{htr})$$

Where:

W_j = specified joint width

W_{\min} = minimum tolerable joint width

w_{tl} = width tolerance of left element

w_{tr} = width tolerance of right element

e_{htl} = horizontal erection tolerance of left element

e_{htr} = horizontal erection tolerance of right element

Using the same logic, the joint width tolerance would be the half the width tolerance of the two adjacent elements plus the erection tolerance of the two adjoining elements.

T_{jw} = Joint width tolerance

$$= \pm (1/2 * (w_{tl} + w_{tr}) + (e_{htl} + e_{htr}))$$

Using these equations, the joint width shown on the plans should be the specified joint width \pm the joint width tolerance. For example: $1\frac{1}{2}'' \pm \frac{3}{4}''$.

Half of the width tolerance is used for each element assuming that the over width would be taken up half on each side of the element.

The PCI Northeast Bridge Technical Committee has developed recommended element fabrication tolerances and element erection tolerances. The committee has also developed details for typical joints based on these equations. **See Sheet SUB-10.**

4.3.2 Horizontal Joints

(Ref: **PCINE Sheet SUB-6**)

The width of horizontal joints is slightly different than vertical joints. Unlike horizontal layout measurements, vertical layout is not normally based on offsets from working lines. This is due to the fact that most bridge structures are made up of only a few stacked elements. The more common way to specify vertical layout is to specify the top elevation of the element within a specified elevation tolerance. When the element is set, the top of the upper element will be checked for elevation. The thickness (width) of the lower connection is adjusted to accommodate the elevation tolerance of the lower element and the height of the upper element.

The thickness (width) of horizontal joints needs to account for the following:

1. The possibility that upper elements will be fabricated taller than detailed, but within the specified element tolerance.
2. The possibility that the lower element will be erected too high but within the specified elevation tolerance.
3. The possibility that the upper element will be erected too high but within the specified elevation tolerance.
4. The minimum thickness (width) of the joint must accommodate the joint filler material after all tolerances are accounted for.

Based on this, the minimum required joint width specified on the plans would be the absolute minimum tolerable joint width plus the total height tolerance of the upper element plus the vertical erection tolerance of the upper element. The total height tolerance is used since the lower joint will be required to accommodate the entire over-height tolerance of the upper element.

The equation for the specified horizontal joint thickness would be:

$$T_j = T_{\min} + h_{tu} + e_{vtl} + e_{vtu}$$

Where:

T_j = specified joint thickness

T_{\min} = minimum tolerable joint thickness

h_{tu} = height tolerance of upper element

e_{vtl} = vertical erection tolerance of lower element

e_{vtu} = vertical erection tolerance of upper element

Elevation check should be specified and measured at the center of the connection, not necessarily all areas of the top surface. For instance, if a footing is to support a column, the elevation tolerance only needs to be checked at the center of the column, not the corners of the footing.

The PCI Northeast Bridge Technical Committee has developed recommended element fabrication tolerances and element erection tolerances. The committee has also developed details for typical joints based on these equations (**see Sheet SUB-6**).

Using the same logic, the joint thickness tolerance would be the height tolerance of the upper elements plus the vertical erection tolerance of the two adjoining elements.

$$T_{jt} = \text{Joint thickness tolerance} \\ = + (h_{tu} + e_{vtl} + e_{vtu})$$

Using these equations, the joint width shown on the plans should be the specified joint width \pm the joint width tolerance. For example: $1\frac{1}{2}" \pm \frac{3}{4}"$.

4.4 Structural Joints

4.4.1 Moment Connections

Elements can be connected with a joint that can transmit moment and shear using the following methods:

4.4.1.1 Embedded Mechanical Splicers

(Ref: PCINE Sheet SUB-5)

The recommended connector for mild reinforcing is a grouted mechanical splicer. **Figure 4.4.1.1** shows the connection of a precast wall panel with a precast footing made with embedded mechanical splicers.

Grouted mechanical splicers are larger than the connected bar, therefore the reinforcing bars must be placed farther from the face of the element in order to provide proper cover over the splicer.



Figure 4.4.1.1 Wall Panel Installation with Mechanical Splicers

These devices can develop 125% of the specified minimum yield strength of the bars (Type 1), or 100% of the specified minimum tensile strength of the bar (Type 2). See ACI 550.1R-01, "Emulating Cast-in-Place Detailing in Precast Concrete Structures." For grouting sequence see Section 5.2.1.3.

One option for testing and acceptance criteria for grouted mechanical splice connectors is ICC Evaluation Service Inc., using their specification AC133 titled "Acceptance Criteria for Mechanical Connector Systems for Steel Reinforcing Bars."

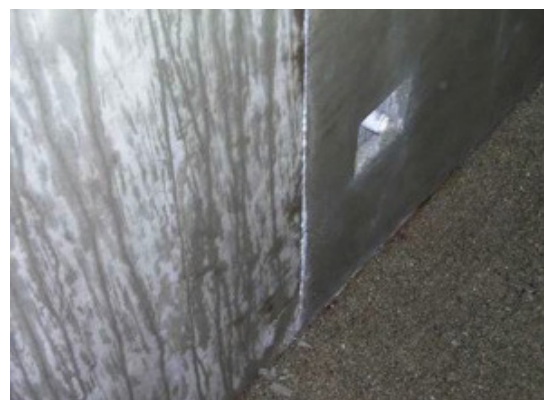
The dimensions of grouted mechanical splicers vary by manufacturer. **Sheet SUB-5** includes dimensional guidelines that can be used for detailing of elements with splicers (based on a review of the three manufacturers that are currently supplying product).



Separate unit



Placing units together



Units connected

Figure 4.4.1.3-1 Example of sections that were match cast for a tight fit. The right photo shows the sections held together by the use of an epoxy adhesive prior to the application of any post tensioning or grouting of connecting reinforcing.

4.4.1.2 Cast-In-Place Closure Pours

(Ref: PCINE Sheets SUB-2, 3, 4, 7 & 9)

There are several methods available for splicing reinforcing within the closure pour including lap splices and mechanical couplers.

Closure pours are also effective; however speed of construction is compromised. This is often used for horizontal moment joints.

In some cases, the closure pours are not required to resist significant forces until the structure is complete; therefore the strength gain and curing of these closure pours may not be on the critical path for the overall construction of the bridge. For example, partial precast footings can be designed to support the dead weight of the bridge, and the combined footings and closure pours can be designed to resist live load and seismic forces. **Figure 4.4.1.1** shows a partial precast footing. The closure pour is located between the two adjacent footings.

4.4.1.3 Post Tensioning with Match-cast Elements

Post Tensioning may be used for complex structures (tall piers), or to eliminate closure pours for horizontal moment connections (integral abutment stems, pier caps, etc.).

In these cases the elements are match cast against each other during production and an epoxy adhesive is placed between the elements during installation. **Figure 4.4.1.3-1** shows the construction of an integral abutment wall stem built with post-tensioning and match cast joints.

The designer shall address shear transfer through match cast moment connections.

Shear transfer can be accommodated by the use of grouted shear keys within the joint, keyed pockets, or by providing additional reinforcement across the joint (shear friction design).

4.4.2 Shear Connections

Certain elements may need to be connected with a joint that only transmits shear using the following methods:

4.4.2.1 Vertical Grouted Keys in Wall Panels

(Ref: PCINE Sheet SUB-10)

Vertical shear joints are typically used in tall vertical wall joints. The purpose of the keys is to limit the differential lateral movement of the wall stem. **Figure 4.4.2.1-1** shows the construction of a cantilever abutment with vertical grouted keys.



Figure 4.4.2.1-1 Vertical Joints in Abutment Wall Stem

4.4.2.2 Horizontal Joints in Wall Panels

Horizontal joints in wall panels are not recommended due to the complexity of the reinforced connections.

4.4.2.3 Grouted Keys in Footings and Approach Slabs

(Ref: PCINE Sheet SUB-10)

Horizontal shear key joints can be used between spread footing elements and approach slab elements. The purpose of the keys is to limit the differential settlement between the adjacent elements. **Figure 4.4.2.3-1** shows construction of precast footings with grouted keys.

Vertical joints between wall panels are analogous to contraction and/or expansion joints in cast-in-place concrete walls. The *AASHTO LRFD Bridge Design Specifications* Article 11.6.1.6 limits the maximum spacing of joints in walls. There is no limit on the minimum spacing of these joints, which justifies the treatment of wall panel joints as contraction or expansion joints.

The joints shown on the PCI Northeast typical detail drawings show various configurations of wall joints corresponding to the equivalent contraction and expansion joints shown in state bridge manuals for cast-in-place concrete (see Sheet SUB-10). These joints do not need to be reinforced since they emulate an unreinforced contraction or expansion joint.

The joints in wall panels can be sized to allow for placement of concrete in place of non-shrink grout.

It is possible to detail a wall panel with horizontal joints; however the difficulty and cost of the detail makes them prohibitive. If wall panel weights get excessive, designers should consider reducing the width of the panel or adding corrugated pipe voids to reduce the element weight.

The joints in footings and approach slabs can be sized to allow for placement of concrete in place of non-shrink grout.



Figure 4.4.2.3-1 Precast Footings Construction

4.4.2.4 Reinforced Dowels

Shear transfer can be developed by means of steel reinforcing bars embedded in a void or pocket.

The void can be formed or made up of an embedded pipe or sleeve. The resistance of the connection can be calculated using interface shear theory (shear friction).

4.4.3 Corrugated Metal Pipe Void Connections for Pile Connections

(Ref: PCINE Sheets SUB-7 & 8)

Corrugated metal pipes (CMP) can be used to reduce element weight; however they can also be used to connect various elements. Reinforcing can be placed in the void and extend into the adjacent element.

CMP voids can be used to connect precast integral abutment stems to piles and drilled shafts. **See Detail 4 on Sheet SUB-8. Figure 4.4.3-1** shows the construction of an integral abutment using CMP voids.

The size of the void should account for pile installation tolerances. **See Detail 5 on Sheet SUB-1.**

In some cases, the vertical pile load needs to be transferred into the precast stem element (when the pile is not directly under each beam). This can be accomplished by using the corrugations to transfer the force via shear. One way of accounting for this is to use punching shear theory, assuming the corrugations are equivalent to a cracked surface.

The connection for pile supported footings can be achieved by providing a blockout or recess in the precast element. CMP voids are an economical means of providing the blockout for the pile. The corrugations can resist both downward and uplift vertical forces through punching shear.

An example of this would be an approach slab to abutment connection.

*Research has shown that CMP void connection acts in a true emulative fashion (i.e. they do not affect the structural behavior of the stem). Based on this, standard integral abutment reinforcing and details (such as pile embedment) can be used.

* *Wipf, Klaiber, and Hockerman, Precast Concrete Elements for Accelerated Bridge Construction, IHRB Project TR-561, Iowa Highway Research Board and the Iowa Department of Transportation, January 2009.*

Tolerance notes on PCINE Sheet SUB-1 details include recommendations for determining the minimum size of the CMP void. The notes are based on a lateral pile installation tolerance of $\pm 3"$ in plan. This is less than typical specification requirements, but has been found to be acceptable when pile driving templates are used. If this cannot be accomplished the size of the CMP void should be increased in order to accommodate the anticipated tolerances. An additional 1" is added to the pile tolerance to allow room for placement of the void concrete.

Punching shear theory is based on limiting the shear stress across a shear plane. The assumed shear plane can be the inside face of the CMP void running from crest to crest of the corrugations. The *AASHTO LRFD Bridge Design Specifications* (Article 5.13.2.5.4) define the nominal punching shear resistance as $0.125(f'_c)^{1/2}$. This equates to 0.25 ksi for fill concrete that has a compressive strength of 4 ksi.



Figure 4.4.3-1 Precast Integral Abutment Construction

4.4.3.1 Corrugated Metal Pipe Void Connections for Wall Cap Connections

(Ref: *PCINE Sheet SUB-6*)

Corrugated metal pipe voids can be used to provide nominal capacity moment and shear connections between wall panels and a precast concrete cap. This can be accomplished by placing reinforcing bars within the void concrete at the interface area between the two elements.

4.4.4 Deck Panel Connections

Connections of full-depth deck panels are generally split into two categories; connections between adjoining panels, and connections of panels to the beams and girders. These are described below:

4.4.4.1 Panel to Panel Connections

There are two categories of deck panel connections. Longitudinal joints typically run parallel to the beam framing. Transverse joints typically run perpendicular to the beam framing, or along the bridge skew.

Longitudinal joints are necessary to accommodate a change in the cross slope, or when the panel installation is phased. Since this connection is typically made in the direction of the primary forces (i.e., the design reinforcing is perpendicular to the beams), the joint is designed to resist these applied forces. These connections are typically field cast closure pours between the panels. Mild reinforcing is extended from ends of each panel and a cast-in-place pour made to connect the panels. It is important to properly develop the projecting reinforcing in the closure pour area. This can be done with lapped bars, hooked bars, or headed reinforcing bars.

Transverse connections are necessary to provide continuous action between panels. The joint should be capable of transferring shear and bending moment between adjacent panels. The *AASHTO LRFD Bridge Design Specifications* recommends the use of grouted shear keys that are subsequently post tensioned longitudinally between end closure pours.

This connection is easier and less costly when compared to a connection made with grouted mechanical splicers. CMP voids allow for larger element and installation tolerances in the wall panels.

The design of the connection can be made using standard reinforced concrete bending and shear theory.

The PCI Northeast Bridge Technical Committee has published details and guidelines for the design and detailing of full-depth deck panels (PCINER-11-FDDP, 2nd Edition, 2011). The document includes information on joints and connections. The following sections outline the typical types of connections used in deck panel bridges.

Some state agencies have tested and used ultra-high performance concrete (UHPC) joints. The UHPC joints typically consist of straight or hooked reinforcing bars extending a short distance from the face of the panels into the UHPC to make the cast-in-place closure pour.

The *AASHTO LRFD Bridge Design Specifications* Section 9.7.5 recommends that the minimum average effective prestress force not be less than 250 psi (applied to the contact area of the joint).

The most common details make use of full depth female-to-female type joints. The joint has a wider opening at the top and nearly closed at the bottom with a polyethylene backer rod to form the bottom of the shear key. The joints are post tensioned longitudinally at mid depth.

4.4.4.2 Connections of Full-Depth Deck Panels to Beams and Girders

Almost all bridges are designed for composite action between the beam and the deck. For full-depth deck panels, composite action is achieved by incorporating welded shear studs (steel beams) or projecting reinforcing (concrete beams) into blockouts or shear pockets detailed into the precast panels.

4.4.5 Connections between Adjacent Beams with Integral Full-Depth Decks

NEXT-D beams and deck bulb tee beams include full-depth integral decks. These beams require a connection between the adjacent deck flanges. This connection needs to be designed to resist the bending and shear forces that are generated by truck wheel loads.

The deck flange connection is normally made using a grouted or concreted closure pour that includes reinforcing steel.

The design of the connection can be based on the same design principles as a cast-in-place concrete deck. The most common design method is the “strip method” as defined in the *AASHTO LRFD Bridge Design Specifications* (Article 4.6.2.1).

There are various methods to detail a closure pour between adjacent beams. Each method makes use of reinforcing steel combined with either grout or concrete that resists the forces acting on the deck at the connection. Several different closure pour connection details have been developed and implemented in the United States. Some connections are based on normal reinforcing bar development equations that are included in the *AASHTO LRFD Bridge Design Specifications*. These include:

1. Lapped reinforcing bars with concrete – This connection uses standard lapped bar details. The concrete can be normal concrete or high early strength concrete depending on the schedule for the project.
2. Hooked reinforcing bars with concrete – This connection is similar to the lapped reinforcing bar detail except standard hooked bars are used to reduce the width of the closure pour.

Other connections are based on recent research that was based on reducing the width of the closure pour joint. These include:

1. Headed and hooked reinforcing bars with high-strength grout or high-strength concrete: Research has shown that headed and hooked reinforcing steel bar splices can be developed in high-strength concrete joints by using short non-contact lap splices (Cast-in-Place Concrete Connections for Precast Deck Systems, *NCHRP Web only Document 173 (Final Report for Project 10-71)*, National Cooperative Highway Research Program, 2011).

2. Straight and hooked reinforcing bars with Ultra-High Performance Concrete (UHPC): Initial research has shown that reinforcing bars can be connected using short lap splices of hooked and straight bars placed in UHPC joints (Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading, *U.S. Department of Transportation, Federal Highway Administration, FHWA Publication No. FHWA-HRT-11-023*).

4.5 Non-Structural Joints

Typical non-structural joints include expansion joints and contraction joints. These are joints that do not have reinforcing passing through them and are not counted on to transfer forces across the joint. **Figure 4.4.2.1-1** shows construction of a precast abutment with vertical contraction joints.

In most cases, these joints should be sealed to prevent moisture from penetrating the area between elements where freezing action could spall the adjacent elements. In some cases, the joints can be left open.

Examples of non-structural joints include retaining wall expansion and contraction joints, joints between different substructure units (abutment to wingwall interface), and joints in long pier bents (where effects of thermal movement can cause large internal frame forces).

Non-structural joints may also be desirable between substructure sections that may potentially experience differential settlement. An example of this would be the interface between a pile supported integral abutment and a long u-shaped wingwall supported on spread footings.

Sealing of the joint can be accomplished by injecting a foam sealant in the opening. The rear face of the wall may be sealed with a membrane sheet, however foam fill is recommended near the ground line or water line. Grouting is also an acceptable option. **See Section 5.3.**

Section 5: Grouting

Guidelines

5.1 Sub-Footings

Cast-in-place (CIP) concrete sub-footing may be used for construction on uneven surfaces, such as bedrock. Sub-footings are used to provide a relatively flat work surface for the placement of precast footings. Individual sub-footings may be used at each leveling screw location as opposed to a continuous full width sub-footing.

The sides of the sub-footing pours need not be formed. The concrete may be cast against the excavation.

5.2 Fill under Spread footings

(Ref: PCINE Sheet SUB-7)

The area below a precast footing needs to be filled with a material that can transfer the footing bearing pressures to the sub-grade below. Bearing pressures under footings are relatively low, in the range of 25 psi to 75 psi. It is recommended that flowable fill be placed under footings, which is a relatively low strength mix of sand, water and cement that can fill the void below the footing, while providing adequate strength. Grouts can be used if high early strength gain is required; however the cost is higher than flowable fill.

Placement of the fill material can be accomplished by pumping or pouring into spaced vertical holes within the precast footing starting from the center and working toward the extremities.

After the fill material has reached a compressive strength that is sufficient to support the footing, the leveling screws, if used can be removed and the fill holes can be grouted.

See Section 3.1.2.5 for more information on grouting under footings.

Commentary

The sub-footing material should be designed to support the loads of the leveling screws. Steel plates can be used to accommodate the leveling screw point loads.

See Detail 4 on Sheet SUB-7 for a suggested Leveling Screw Detail.

The bearing pressure noted corresponds to roughly 4 ksf to 11 ksf.

It is recommended that the spacing of the holes be no greater than 6 feet. If pressure grouting is the chosen method of filling this void, the operation must be monitored to ensure the footing is not lifted. A dam around the perimeter of the footing should be used to create a hydraulic head which will facilitate the flow of grout and retain the fill material.

5.3 Element to Element grouting

It is the contractor's responsibility to determine the specific type of grout to be used in each joint, and the methods of installation based on the notes on the plans and in the specifications.

A prepackaged, shrinkage-compensating, flowable grout is recommended for most connections. The strength of the grout should be equal to or greater than the strength of the joined elements.

5.3.1 Horizontal Surfaces

5.3.1.1 Recessed Key Connection

(Ref: PCINE Sheet SUB-5, 9 & 10)

This joint is typically found at the stem/footing joint in abutments. Use of a recessed key will improve the shear capacity and will create adequate head to help push a flowable grout through the joint minimizing the need to pump the grout into place. **Detail 4 on Sheet SUB-9** shows an example of the recessed key.

The recommended grouting procedure for recessed key connections is as follows **(see Detail 6 on Sheet SUB-5)**:

Step 1: Fill the key to just below the lower port of the grouted mechanical splice. A recessed keyway or grout dam may be used to create a hydraulic head for grout flow. The grout may be installed by pouring the grout into the keyway slot or behind the grout dam until the inlet side will accept no more grout. This procedure should be started at the end of the joint and proceed continuously along the joint.

The grout placed in step 1 shall be kept out of the mechanical splice by the use of a washer or stopper that is supplied by the mechanical splice manufacturer.

The assembly plan developed by the contractor should specify the type of grout and method of installation for each joint. **See Section 7.3.**

The designer should include a note on the plans or in the specifications describing the required properties of the grout in each connection.

The grout within this joint must transfer the compressive load of the stem-to-footing moment couple in resisting overturning loads in the connection. Grout also can provide some measure of corrosion protection for the connections within the joint. This joint is typically found in horizontal connections between elements where there is significant load transfer. This grout may take many forms including prepackaged grout, dry pack, pre-placed prepackaged mortar (battered), or grout placed under pressure.

The use of prepackaged grout is recommended. The compressive strength of the grout should meet or exceed the strength of the concrete in the wall and footing.

Filling this joint from both sides and both ends simultaneously will increase the chance for a void within the key. Pumping the grout into place should be encouraged as it supplies a continuous flow of grout making it easier to maintain a continuous flow through the key.

Washers placed over the rebar extensions provide the seal to keep the mechanical splice free of step 1 grout.

Step 2: Grout the mechanical splice per manufacturer's recommendations.

Step 3: Fill the remainder of the key.

The recommended grouting procedure for non-recessed key connections is shown on **Detail 5 on Sheet SUB-5**.

The grout placed within this type of joint should also provide protection from the elements and as such, if the joint is open to the weather, water shedding detailing should be employed.

Other methods of grouting have been used successfully. The project specifications should allow for contractor alternate grouting procedures. The procedures should ensure that the key grout does not flow into the grout mechanical splice.

5.3.2 Vertical Joints

(Ref: PCINE Sheet SUB-10)

A flowable, cementitious grout should be used for vertical joints. It should be introduced at the top of the joint, filling it from bottom to top.

Designers should specify the use of rigid formwork for the joints and rodding of grout during installation to minimize voids.

If shear transfer is not required, consider filling this joint with expanding foam sealant or other fillers.

Pre-applied rigid joint filler materials are not recommended. Inserting rigid fillers after assembly is also not recommended.

This connection does not use a key. If shear and/or compression transfer is required through the grout. The grout should be a non-shrink material.

The designer should choose the most appropriate type of grout for the anticipated exposure conditions.

This joint is typically found between vertical wall elements. Significant hydraulic head will be created due to the typical height of the joints being filled. Backer rods placed at the edges of the enclosed vertical joint as a dam against the fluid grout will not be adequate in restraining the grout due to the fluid pressure. It is recommended that such a joint be restrained with formwork in most cases. The grout should be rodded during placement to ensure that voids are eliminated.

This treatment may be considered adequate if the joint is deemed non-structural. The expanding foam keeps the joint free of foreign material and should be supplemented with a flexible joint sealant (both sides) and membrane on the fill side for waterproofing.

Experience has shown that tolerance between the elements will be compromised, which makes element assembly virtually impossible. Installation of fillers after assembly results in a poor quality joint.

5.3.3 Mechanical Grouted Splices

(Ref: *PCINE Sheet SUB-5*)

Sheet SUB-5 (Detail 8) contains information on the installation and grouting of Mechanical Grouted Splices.

Only grout specified by the mechanical splice manufacturer should be used.

The type of grouting operation is usually dependent on the orientation of the sleeve. A training session on proper grouting techniques is recommended for field personnel.

The grout used in the sleeves is part of the splice “system.” The grouts used in the sleeves must be the same grout used for certification tests.

A sleeve cast in the upper element is a post-grouted connection. Manufacturer recommendations shall be followed.

The manufacturer will typically require special equipment to mix and install the grout.

A sleeve cast in the lower element can be filled with grout from the top and the ports are plugged after the sleeve has been purged of air. The upper element is lowered into position and the bars extending from the element are pushed into the sleeve displacing the grout into the surrounding joint. Sleeves in this position can also be grouted after setting the elements using ports.

The text above is for typical installations. The sleeves are proprietary and each company will have different variations for these grouting procedures.

Experience has shown that the grouting procedure is easily mastered by construction crews after a few sleeves. Based on this, it is recommended that design engineers specify a test mock-up connection with a minimum four sleeves.

These connections are typically at integral abutment caps, pile bent caps, or pile supported footings. Self-consolidating concrete can be used for caps with small fill and vent ducts to ensure adequate consolidation without segregation.

5.4 Pile and Wall Caps

(Ref: *PCINE Sheet SUB-6, 7 & 8*)

Self-consolidating concrete is recommended to fill the void. The concrete should either have limited shrinkage characteristics or be made with a shrinkage-compensating admixture.

The concrete should be placed through fill ducts into pile blockouts. Vent ducts shall also be provided into the blockout. When fill and vent ducts are used they should be corrosion resistant (PVC or galvanized steel). At least one fill and one vent duct should penetrate into each pile blockout. **See Detail 4 on Sheet SUB-6.**

5.5 Post-Tensioning Ducts

Grouting of post-tensioning ducts should be done using a grout designed for pressure grouting the annular spaces around post-tensioning bars or cables.

Post-tensioning ducts are proprietary items designed to fit a particular post-tensioning system. The system chosen must fit within the construction and do so within normal tolerances.

Special care should be taken for grouting of ducts in either vertical or sinusoidal patterns.

Having two ducts per blockout allows for concrete to be placed in one duct, while placement is being monitored in the other duct.

There are several commercial grouts available that are designed for this purpose. These grouts have been designed to be pumped through small annular spaces over long distances without segregating. If a grout is used that has not been designed for this purpose it is likely that the aggregates will segregate and result in plugging the grout ports, lines or pump, and possibly compromising the grouting operation.

Using a corrugated duct in combination with a grout allows the post-tensioning tendon to become developed along its length in the event of any loss of end anchorage due to corrosion. The designer should consult with the grout manufacturer to select a duct size that is consistent with the recommended grout. In most applications allowing a total of $\frac{3}{4}$ " of tolerance should be adequate. (For example, using a 3-inch duct for a $1\frac{3}{8}$ " post-tensioning bar)

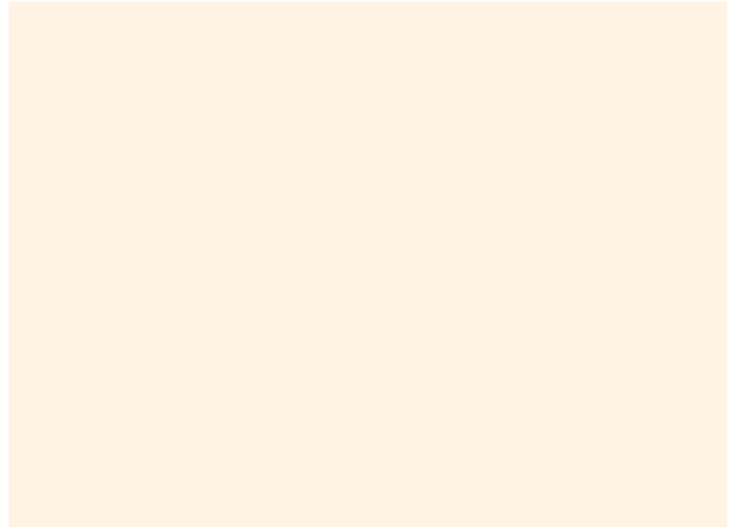
There have been problems with excess bleed water in post-tensioning ducts that have led to severe corrosion of the tendons. The following publications are recommended resources for more information on the proper installation and grouting of post-tensioning systems:

FHWA, "*Post-Tensioning Tendon Installation and Grouting Manual*," (FHWA-NHI-13-026, May 2013)

Post Tensioning Institute, PTI M55.1-12: *Specification for Grouting of Post-Tensioned Structures*.

5.6 Blockouts for Anchoring Devices

Blockouts are used to recess bolting mechanisms such as those for post-tensioning strands in butted beam decks or for the anchoring bolts in precast rail. All open blockouts on the structure should be filled with a prepackaged non-shrink grout specifically formulated for the intended orientation of the work (horizontal, vertical, or overhead). First ensure the recess is free of dust and other construction debris by cleaning the area according to the grout manufacturer's recommendations. Apply the grout using a trowel into the recess in layers to ensure the cavity is completely filled. The final layer shall be troweled smooth with the face of the element. The grout color and texture shall closely match the element.



Section 6: Seismic Considerations

Guidelines

6.1 General Criteria

In general, the design and details of precast concrete elements for seismic forces should be consistent with cast-in-place concrete construction.

6.2 Connection of Superstructure to Substructure

There are several methods of transmitting seismic horizontal loads from the superstructure to the substructure. In most cases this consists of designing the connection between the stringers to the substructure to also resist the seismic horizontal loads. This section discusses some of the other options that are typically used.

6.2.1 Keeper Blocks

Keeper blocks consist of reinforced concrete blocks that are placed between two interior beams to transmit lateral forces from the superstructure to the substructure. Keeper blocks are often the most cost-effective means of restraining a bridge for seismic events.

Bridges should be designed with only one keeper block per superstructure unit.

Commentary

All provisions specified in AASHTO need to be satisfied in a precast concrete bridge. This includes but is not limited to reinforcing steel in footings, column confinement, and connections between the superstructure and substructure.

The process of designing seismic reinforcement is the same as for a cast-in-place concrete structure. Slight variations in detailing may be required because of the use of precast elements. These issues are covered in this section.

In many states, the connection of the superstructure to the substructure is detailed as a pinned connection. Integral (moment) connections are also commonly used.

There are other options for seismic restraint such as cable restrainers and seismic isolation devices. These methods are also acceptable; however they are not included in this document.

Keeper blocks are used to resist lateral seismic forces. Longitudinal seismic forces can be resisted by the abutment backwall, or anchor rods.

Concrete keeper blocks have very low ductility. If two blocks were detailed, it is likely that one would carry all the seismic demand. Distribution of seismic forces to other keeper assemblies would most likely only occur after failure of the first keeper.

If keeper blocks are precast, the connection of keeper blocks can be made using several of the methods outlined in this document.

Consideration should be given for casting keeper blocks in place. **See Details on Sheets SUB-2, 3 & 4**

6.2.2 Pilasters/Cheekwalls

Pilaster/Cheekwalls are used on adjacent box beam bridges near the fascia beam ends. They resist lateral seismic forces.

The design is similar to keeper blocks. Each pilaster must be capable of resisting the entire lateral seismic force at each substructure unit, since only one pilaster/cheekwall will engage the superstructure at a time.

6.2.3 Abutment Backwall

(Ref: PCINE Sheet SUB-9)

Longitudinal seismic forces can be resisted by the abutment backwall. The design of abutment backwalls for seismic forces is similar to keeper blocks.

Designers should verify that the distance between the backwall and the beam/slab is sufficient for thermal movement, but small enough so that the beams do not slide off the substructures.

6.2.4 Anchor Rods

The design of anchor rods for lateral load should take into account the bending capacity of the rod, edge distance to the concrete foundation, internal reinforcing around the embedded portion, strength of the concrete, and group action of the rods.

Keepers can be installed using grouted mechanical splices, or can be cast integral with the precast substructure element.

Pilasters are usually only used on adjacent box beam bridges since there is no gap between adjacent beams. On stringer bridges, keeper blocks are probably more cost effective, since only one keeper is required to resist lateral seismic forces in both directions.

The commentary from **Section 6.2.1** is also applicable to this section.

The design of seismic restraining systems is based on limited and repairable damage. Using backwalls for restraint will inevitably result in a structure that has shifted longitudinally during the seismic event. The structure may need to be jacked back into position after seismic events.

One of the most important aspects of seismic design is to prevent superstructures from sliding off substructures. This issue becomes more pronounced on multiple span bridges where all joints between spans are assumed to be closed in one direction.

The AASHTO specifications do not specifically address the design of embedded anchors loaded in shear. Designing for the shear capacity of the rod is only part of the potential failure mechanism. The rods tend to fail in combined bending and crushing of the concrete around the rod. The AASHTO code refers to this failure mechanism as a "global" failure. AASHTO Article C14.8.3.1 references the American Concrete Institute publication "*Building Code Requirements for Structural Concrete*" (ACI 318-05), Appendix D, for the investigation of this failure mechanism.

Anchor rods should be designed to be ductile. The use of high-strength, heat-treated rods is discouraged due to low ductility. The embedded portion of the rod shall be properly reinforced in order to prevent brittle fractures of the surrounding concrete.

Material for anchor rods should be ASTM F1554, and should be either threaded (with nuts) or swaged on the embedded portion of the rod. The design yield strength of this material may be specified as 36 ksi (250 MPa), 55 ksi (380 MPa), or 105 ksi (725 MPa), depending on the design. The yield strength should be given in the specifications or on the plans.

6.2.5 Integral Connections

(Ref: PCINE Sheet SUB-8)

Superstructures can be connected to substructures using integral moment connections. In most cases, this connection will be made with a cast-in-place closure pour or by using grouted mechanical splices.

6.3 Column Connections

Columns are often the most heavily loaded elements during a seismic event. Special care shall be taken to properly detail connections in precast column elements.

During a seismic event, it is inevitable that only a percentage of the rods will initially see load due to construction tolerances. Ductility in the rods will ensure that all rods will work together to resist seismic forces.

The anchor rods should normally be surrounded by lateral reinforcing steel near the surface of the concrete. This will allow lateral forces to be resisted after the initial cracking of the concrete.

This material is specifically designed for anchor rod applications. Other materials have been used, but do not offer the economies of ASTM F1554. The designer should offer options of swaging or threading the anchor as different suppliers supply one or both of these options.

The most common form of integral connection is between beams and abutments in stringer bridges. In these cases, the end diaphragm is usually cast in place between the beams due to the complexity of the shapes.

Integral connections have successfully been made between beams and abutments using grouted mechanical splices cast into the beam-ends. It is also common in high seismic regions to make integral connections between the superstructure and piers. These connections are similar to integral abutment connections.

Designers should also refer to individual state requirements with respect to the design of integral abutments.

In high seismic regions, columns are designed to form plastic hinges and contribute to dissipation of seismic forces. The high demand region on a typical column is at the ends where the column connects to the footings and pier caps.

6.3.1 Column Base and Cap Connections

(Ref: PCINE Sheets SUB-5 & 6)

The use of mechanical splices that can develop 100% of the specified tensile capacity of the connected reinforcing bar is recommended for columns in high seismic zones.

There are two levels of mechanical splice devices specified in the ACI 318 Code. Type 1 mechanical splices need to be capable of developing 125% of the specified minimum yield strength of the bar. Type 2 mechanical splices need to be capable of developing 100% of the specified minimum tensile strength of the bar (equal to 150% of specified yield for ASTM A615 bars).

The 125% development is required for all mechanical reinforcing couplers in the AASHTO specifications. This level of strength is not sufficient for plastic hinge zones that will experience significant ductile behavior. The recommendation is based on the requirements of the ACI 318 Code for Special Moment Frames constructed using precast concrete. The designer should specify the strength requirements for the Type 2 connectors in the contract.

According to the current AASHTO code, mechanical splices should be staggered a minimum of 24" for bridges in high seismic zones. (Section 5.10.11.4.1f of the LRFD Specifications). This provision is based on the fact that the AASHTO specifications are limited to Type 1 mechanical splices only.

The current ACI 318 code has similar restrictions for Type 1 mechanical splices; however the use of Type 2 mechanical splices are unrestricted in special moment frames constructed with precast concrete (including the plastic hinge zone). For these reasons, it is recommended that designers follow the provisions of ACI 318, Article 21.8 "*Special moment frames constructed using precast concrete.*" This includes the requirements for determining the length of the plastic hinge region.

One way to avoid having mechanical connectors in the plastic hinge zone is to place all of the mechanical connectors within the footing (see Detail 5 on Sheet SUB-5) and within the pier cap (**see Details 2 and 3 on Sheet SUB-6**) where plastic hinging is not a factor. While this is possible, it makes handling of the column more difficult, which can lead to higher costs.

The AASHTO specifications for low to moderate seismic zones allow for splicing 100% of the longitudinal bars with mechanical splices at one location. High seismic zones are defined as Zones 3 and 4 in the LRFD Specifications.

6.3.2 Splices along Column Length

The connection of column-to-column splices is similar to that used for column-to-footing and column-to-cap connections.

6.3.3 Confinement Reinforcement

(Ref: *PCINE Sheet SUB-5*)

It is possible to provide confinement for vertical reinforcing in precast concrete columns. AASHTO provisions for confinement based on cast-in-place concrete construction should be followed.

Vertical bars can be confined with transverse ties detailed in accordance with the *AASHTO LRFD Bridge Design Specifications*.

Transverse ties need to be properly detailed in order to achieve confinement. These provisions require continuous “confinement,” not necessarily continuous “reinforcing” across the joint between the column and pier cap or footing. It is acceptable to properly terminate the transverse tie confinement steel at the end of the column, and also in the adjoining element. Refer to the AASHTO LRFD Specifications for the latest provisions regarding this issue.

Another approach that is acceptable is to detail individual hoops detailed according to the *AASHTO LRFD Bridge Design Specifications*. The ends of the hoop bars need to be properly embedded into the core of the column.

A third approach is to use welded hoop ties. The welding process needs to be prequalified. The California DOT has a prequalification testing specification and a list of prequalified fabricators. Designers should not use these types of ties without coordinating with the approving agency.

6.4 Footings

The design of precast footings for seismic forces should follow normal procedures for cast-in-place concrete footings.

6.4.1 Internal Reinforcement

Column confinement reinforcement shall extend into the footing as noted in the AASHTO design specifications.

This reinforcement refers to the confinement of the dowel bars in the footing that connect with the longitudinal column bars above.

Refer to the discussion in **Section 6.3.3**.

6.4.2 Pile Connections

(Ref: *PCINE Sheet SUB-7*)

Special details are required to provide pile uplift capacity in precast footings. Several methods are available:

Reinforcement from concrete piles (precast or cast-in-place) can be extended into pockets in the precast footing. This reinforcement can also be installed on the top of the pile by drilling and grouting.

Weldable steel reinforcement can be welded to the top of steel piles after cut-off. These bars can be extended into pockets in the precast footing.

Pockets for piles should be formed with corrugated metal pipes. The corrugations can transfer the pile forces into the footing via shear friction action. The pockets should extend through the footing to facilitate concreting and/or grouting.

Reinforcement for concrete pile uplift should be similar to cast in place footing construction.

It is important to use weldable reinforcement when making welded connections.

Pockets formed with corrugated metal pipes have been successfully used for the connections between integral abutment stems and piles. The forces acting on a pile supported footing are similar. The corrugations can transfer uplift and downward forces. AASHTO LRFD Section 5.8.4 should be used to check the shear capacity at the pipe interface. The failure plane for this mode of shear is defined as the shear plane across the valleys of the pipe. This equates to area defined by the innermost circumference of the pipe multiplied by the depth of the footing.

The pocket size should be just large enough to account for the size of the pile adjusted for the anticipated pile placement tolerances (**see Detail 5 on Sheet SUB-1**).

Section 7: Fabrication/Construction

Guidelines

7.1 Lifting Devices

The location and design of lifting devices is the responsibility of the precast manufacturer.

Locations that are visually sensitive should be identified on the contract drawings. Lifting devices should not be used in these areas if possible. Lifting devices in these areas need to be recessed, easily removed, and patched to approximate the color and texture of the surrounding concrete.

7.1.1 Corrosion Protection

Provisions shall be made to protect the device from corrosion when the device is to be exposed to the environment in the finished construction.

7.2 Equipment

7.2.1 Shipping and Handling

The size of precast elements should be finalized by the precaster and contractor with consideration for shipping restrictions, equipment size and availability along with site constraints. The final element sizes will be shown on the assembly plan.

7.2.2 Slide-in Bridge Construction (SIBC)

In certain situations, it may be desirable to assemble the entire structure or a portion of the structure adjacent to the final installation location and move it horizontally into its final position.

Commentary

If possible, lifting devices should be on the back face or top surface of elements, hidden from view.

While some lifting devices may be located in areas that will be hidden, most will need to be removed. This will likely result in a portion of the lifting device being exposed. The exposed steel will in time allow corrosive materials to leach into the concrete. To prevent this, the contractor should apply a patch to seal the exposed steel from corrosion. The lifting device may need to be partially removed to provide adequate cover in the patch area.

Lifting devices that will remain in place in highly corrosive environments (such as parapets) may require the use of galvanized steel.

Most elements can be shipped on flat bed trailers. Unusual trailer configurations and support frames should be avoided unless the quantity of pieces justifies the special equipment.

Structures with weights as high as 6 million pounds can be moved into place using specialized equipment such as skid rails with hydraulic jacks, rollers, modular transporters (self-propelled or towed), or cable systems.

This is complex and specialized work; therefore it should only be used in areas where the time of change-out from an old structure to a new structure is limited.

7.3 Assembly Plan

This plan is created by the precaster and contractor and submitted to the owner and/or the engineer of record for approval. It provides detailed information on the contractor's means and methods for assembling the elements.

The assembly plan should at the very least, include all information required to complete the work such as:

- Engineer of record for the assembly plan.
- Shop drawings of all elements.
- Specific product names and other material requirements for all proprietary products proposed for use.
- Data on all materials that are the responsibility of the contractor.
- Details of all equipment to be used to lift elements including cranes, excavators, lifting slings, sling hooks, and jacks. Include crane locations, operation radii, and lifting calculations.
- Work area plan depicting items such as utilities overhead and below the work area, drainage inlet structures, and protective measures.
- Temporary support requirements for substructures including leveling screws and/or shims and lateral load and moment resistance requirements for vertical elements during assembly. Include methods of adjusting and securing the element after placement.
- A detailed sequence of construction and a timeline for all operations. Account for setting and cure time for grouts, grouted splice couplers, and concrete closure pours.
- Procedures for controlling tolerance limits both horizontal and vertical. Include details of any alignment jigs including bi-level templates for reinforcing anchor dowels.
- A detailed installation procedure for connecting the grouted splice couplers (if required) including pre-grout and post-grout applications.
- A list of personnel that will be responsible for the grouting of the grouted splice couplers. Include proof of completion of two successful installations within the last two years. Training of new personnel within three months of installation by a manufacturer's technical representative is an acceptable substitution for this experience. In this case, provide proof of training.

The assembly plan is one piece of a project delivery concept devised for accelerated bridge construction. This concept allows the owner to design the structure and gives the contractor the ability to decide the most suitable means to assemble the elements.

The contract drawings provide a design and standard details for joints within the structure and performance requirements for materials that are used to assemble the elements.

Good resource for installation information:

1. PCI MNL-132: *Erection Safety for Precast and Prestressed Concrete*.
2. PCI MNL-127: *Erectors Manual—Standard and Guidelines for the Erection of Precast Concrete Products*.

- Methods of forming closure pours including the use of backer rods. Do not assume that the backer rods will restrain the pressure from the grout in vertical grout joints. Provide additional forming to retain the backer rod.
- Proposed methods and methods for installing non-shrink grout and the sequence and equipment for the grouting operation.
- Methods for curing grout and closure pour concrete.
- Methods for placement of flowable fill for spread footings. Add additional grout ports in the footings to facilitate the bedding process if required.
- Details of post-tensioning tendon manufacturer, installation methods, tendon jacking (including elongations), and duct grouting procedures.

7.4 Coordination

Coordination between all parties is paramount with accelerated construction. Project specifications should include a mandatory coordination meeting prior to the start of construction of precast elements.

A preconstruction/preassembly meeting should be held with all the people involved in the project to discuss the assembly plan and construction of the bridge. A decision making hierarchy should be established in order to address issues as they arise.

7.5 Tolerances

(Ref: PCINE Sheet SUB-1)

All elements are fabricated to a tolerance. There are several different tolerances that can affect the construction of precast elements. Designers should specify tolerances for fabrication and erection of elements.

The following sections contain information on different tolerances. The width or thickness of joints between elements is affected by the tolerances. **See Section 4.3** for information on joint tolerances.

The importance of establishing lines of communication between all parties involved cannot be overstated. The coordination meeting should occur after the approval of the assembly plan.

The decision makers in each discipline must be identified early, and they must be available to make timely decisions when things don't go as smoothly as planned during the assembly process.

Lack of tolerance control is the primary cause of construction problems on bridges built with precast elements. The specification of tolerances in project plans and/or specifications will help to alleviate these problems. If tolerances are not specified, the fit-up of the elements could be compromised.

7.5.1 Fabrication Tolerance

(Ref: **PCINE Sheet SUB-11 & 12**)

All precast elements are manufactured to a tolerance. Designers should include element tolerance details in the plans or specifications.

7.5.1.1 Inserts, Voids, and Projecting Reinforcing

(Ref: **PCINE Sheet SUB-11 & 12**)

The erection tolerance and hardware tolerances are interconnected. If a connection involves the insertion of a reinforcing bar into a device (coupler or duct), the specification for tolerances would be based on the assumption that the bar is installed to one side (say: to the left) and the coupler installed to the opposite side (say: to the right). The combination of these two potential installation tolerances needs to be kept within the tolerance of the insertion of the bar in the device.

The equation for the horizontal location of the specified projecting bar location tolerance would be:

$$T_b = 1/2 * T_{id}$$

Where:

T_b = Specified bar location tolerance

T_{id} = Insertion tolerance of the bar on the device based on the requirements of the manufacturer of the device

The equation for the specified device location tolerance would be:

$$T_d = 1/2 * T_{id}$$

Where:

T_d = Specified device location tolerance

T_{id} = Insertion tolerance of the bar on the device based on the requirements of the manufacturer of the device

7.5.2 Erection Tolerances

(Ref: **PCINE Sheets SUB-3 & 4**)

The erection and setting of heavy precast elements are controlled through the use of erection tolerances.

Designers should include element erection tolerances in the plans. Erection tolerances should be measured from a common working line that is shown on the plans.

The PCI Northeast Bridge Technical Committee has developed recommended tolerance drawings for typical precast elements. Details for typical element tolerances are included on **Sheets SUB-11 & 12**.

Erection of elements based on center-to-center spacing can result in a build-up of erection errors. The use of working lines is critical to prevent this build-up of errors.

7.5.2.1 Vertical Control in the Field

(Ref: PCINE Sheets SUB-2, 3 & 4)

Horizontal joints are used to control vertical geometry by accounting for the element tolerances and erection tolerances so that the final elevations are as specified on the contract plans.

Vertical control is normally established on the top of each element at critical locations, typically at the center of the joint.

7.5.2.2 Horizontal Control in the Field

(Ref: PCINE Sheets SUB-3 & 4)

Vertical joints are used to control horizontal geometry by accounting for the element tolerances and erection tolerances so that the final layout is as specified on the contract plans.

Horizontal control is typically done through the use of working lines. The contractor should survey and layout location of elements based on a common working line prior to installation. Layout control should continue throughout assembly.

7.6 QA/QC

Plant cast prestressed bridge products are produced in PCI certified plants. Products are manufactured in accordance with the *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, Fourth Edition, MNL-116-99.

Errors in horizontal joints can accumulate with each joint if no adjustment is specified. Horizontal joints should be detailed to allow for minor adjustments as required during construction. This is normally done through the use of shims placed in the center of the horizontal joint.

7.6.1 Repair of Elements

As with any manufacturing process defects and damage can occur in precast concrete bridge products. Examples may include voids, cracks, as well as missing, improperly located, or damaged reinforcement and hardware. The repairs of defects and damage should be according to either of the following:

1. *PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, 1st Edition, (MNL-137), located at www.pci.org.
2. *Bridge Member Repair Guidelines*, Report # PCINER-01-BMRG. Located at www.pcine.org.

7.6.2 Post-Tensioning

The inspection of post-tensioning tendon installation and grouting of ducts should be specified in the contract documents.

7.6.3 Mechanical Grouted Splices

A template will be required for accurate mechanical splice placement during element fabrication and/or field cast conditions to ensure element compatibility.

The grouting process should follow the manufacturer's published recommendations for materials and equipment.

7.7 Backfill

The use of backfill materials should be carefully coordinated with the geotechnical engineer for the project.

The PCI Repair Manuals were developed to assist designers in determining acceptance, repair or rejection of the product.

The following publications are recommended resources for more information on the proper installation and grouting of post-tensioning systems:

- FHWA, "*Post-Tensioning Tendon Installation and Grouting Manual*," (FHWA-NHI-13-026, May 2013).
- Post Tensioning Institute, PTI M55.1-12: *Specification for Grouting of Post-Tensioned Structure*.

Templates should be used during fabrication to ensure fit-up between joined elements. Proper dowel extensions are required to develop the full capacity of the grouted mechanical splice. Placement tolerances should be as recommended by the mechanical splice manufacturer.

A minimum of two inspectors should be required for the mechanical splice grouting operation: one to watch the grout preparation and one to watch the grouting process.

7.7.1 Flowable Fill

This material is also known as “controlled density fill.” It has the ability to rapidly backfill a structure without the need for compaction. The designer should investigate the effects of the flowable fill on the substructure.

The designers should consider the effects of shrinkage of thick flowable fill layers during curing.

The designers should consider the cure time and strength gain time when using flowable fill.

7.7.2 Compacted Granular Fill

Normal compacted granular fill may be used for backfilling operations.

Flowable fill can be installed quickly; however it has several drawbacks. The actual material is more expensive than granular fill, and the area to be filled will need to be secured with either formwork or embankments. In some cases, the cost of these items may outweigh the cost of compacting traditional granular fill.

Flowable fill will exert significant fluid pressure on the substructure prior to setting. This loading condition should be checked in the design, if flowable fill is specified.

Flowable fill has been known to shrink significantly when placed in thick layers. The engineer should consider this when placing thick layers of fill. A secondary thin topping pour may be necessary under critical elements. In most cases, a layer that is less than 12 inches should not be problematic (under footings and slabs).

Flowable fill is not a rapid strength gain material. The designer should determine the strength gain characteristics when determining an ABC schedule.

7.7.3 Foam Products

Foam products can be used to facilitate backfilling operations. These products consist of lightweight (3 pcf) polystyrene blocks that are stacked behind a substructure unit.

The designer should investigate the effects of this material on the design of the substructure since the unit weight is much less than traditional granular backfills.

The elevation of the water table should also be studied since these products can float.

Stacking of these blocks can progress very rapidly. These blocks are normally supplemented with flowable fill and/or granular backfill.

The light unit weight may affect the design of structures where the dead load of the backfill is used to counteract overturning forces.

There are two issues with these products. First, the compressive strength of the blocks is limited; therefore a layer of granular material above the blocks will be required in order to distribute the wheel loads to the blocks. Second, the designer should investigate floatation of the blocks in areas where the water table is above the bottom of the blocks. Often the buoyancy can be offset by the weight of the granular fill over the blocks.

Another issue is that polystyrene blocks are highly reactive to petroleum products. In some cases, polystyrene blocks can dissolve rapidly when in contact with petroleum fuels. Some states require the use of membrane systems or concrete slabs over the blocks for this reason.

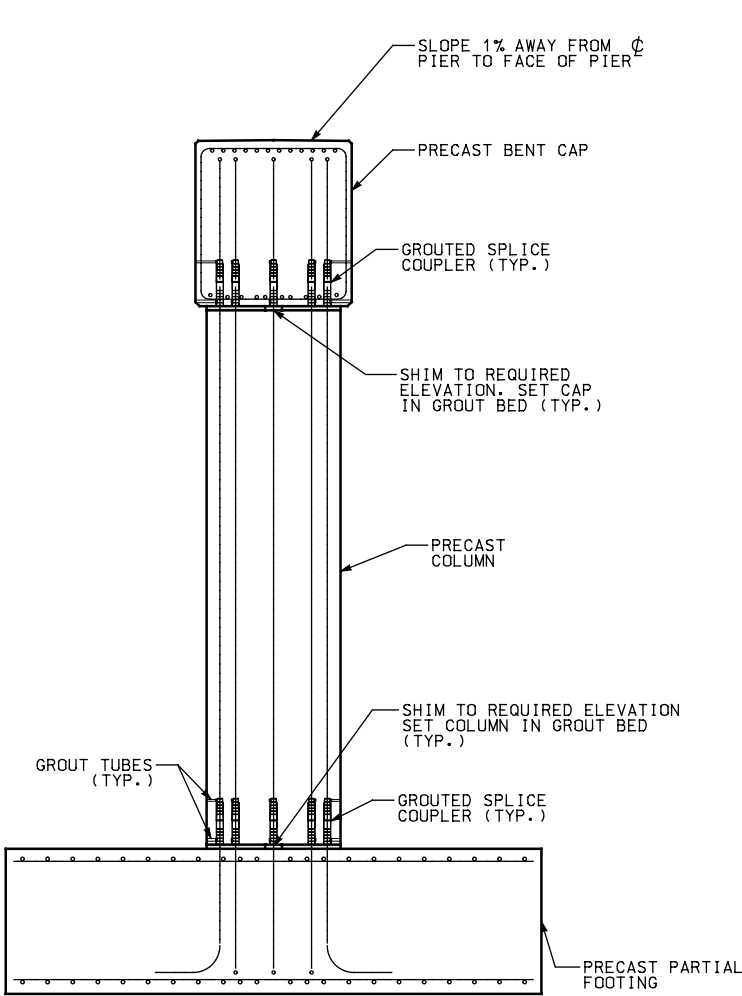
Section 8: References

1. AASHTO LRFD Bridge Design Specifications, 6th Edition with 2013 Interim Revisions, American Association of State Highway and Transportation Officials.
2. AASHTO Standard Specifications for Highway Bridges, 17th Edition, American Association of State Highway and Transportation Officials.
3. PCI Documents (located at www.pci.org)
 - A. *PCI Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products*, PCI MNL-116. Precast/Prestressed Concrete Institute, Chicago, IL.
 - B. *Tolerance Manual for Precast and Prestressed Concrete Construction*, First Edition, MNL 135-00. Precast/Prestressed Concrete Institute, Chicago, IL, 2000.
 - C. *Bridge Design Manual*, Third Edition, PCI MNL-133-11. Precast/Prestressed Concrete Institute, Chicago, IL.
 - D. *State of the Art Report on Full-Depth Precast Concrete Bridge Deck Panels*, PCI Report Number SOA-01-1911, 2011.
 - E. *PCI Design Handbook Precast and Prestressed Concrete*, Seventh Edition, The Precast Prestressed Concrete Institute, Chicago, IL, 2010.
 - F. *PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, 1st Edition, MNL-137. Precast/Prestressed Concrete Institute, Chicago, IL.
 - G. *Precast Prestressed Concrete Piles—Chapter 20, Bridge Design Manual*, 1st Edition, BM-20-04, Sept. 2004.
 - H. PCI MNL-132: *Erection Safety for Precast and Prestressed Concrete*, Precast/Prestressed Concrete Institute, Chicago, IL.
 - I. PCI MNL-127: *Erectors Manual—Standard and Guidelines for the Erection of Precast Concrete Products*, Precast/Prestressed Concrete Institute, Chicago, IL.
4. PCI Northeast Documents (located at www.pcine.org)
 - A. Northeast Extreme Tee Beam Guidelines NEXT Beams (June 2012).
 - B. Northeast Bulb Tee (NEBT) Post-Tensioned Design Guidelines (June 2001).
 - C. Load Charts for Northeast Bulb Tee - LRFD Load Charts (2008).
 - D. Load Charts for Northeast Bulb Tee - Instructions for Use and Section Properties (2008).
 - E. Full-Depth Deck Panels Guidelines for Accelerated Bridge Deck Replacement or Construction (April 2011).
 - F. Bridge Member Repair Guidelines (January 2003).
 - G. Prestressed Concrete Girder Continuity Connection (May 1998).
 - H. Precast Deck Panel Guidelines (May 2001).
5. FHWA, *Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES)*, May 2006, Federal Highway Administration, Washington, D.C.
6. *Connection Details for Prefabricated Bridge Elements and Systems*, U.S. Department of Transportation, Federal Highway Administration, Publication Number FHWA-IF-09-010, March 2009.
7. *Accelerated Bridge Construction - Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems*, U.S. Department of Transportation, Federal Highway Administration, November 2011.
8. *Cast-in-Place Concrete Connections for Precast Deck Systems*, NCHRP Web only Document 173 (Final Report for Project 10-71), National Cooperative Highway Research Program, 2011.
9. *Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading*, U.S. Department of Transportation, Federal Highway Administration, FHWA Publication No. FHWA-HRT-11-023).

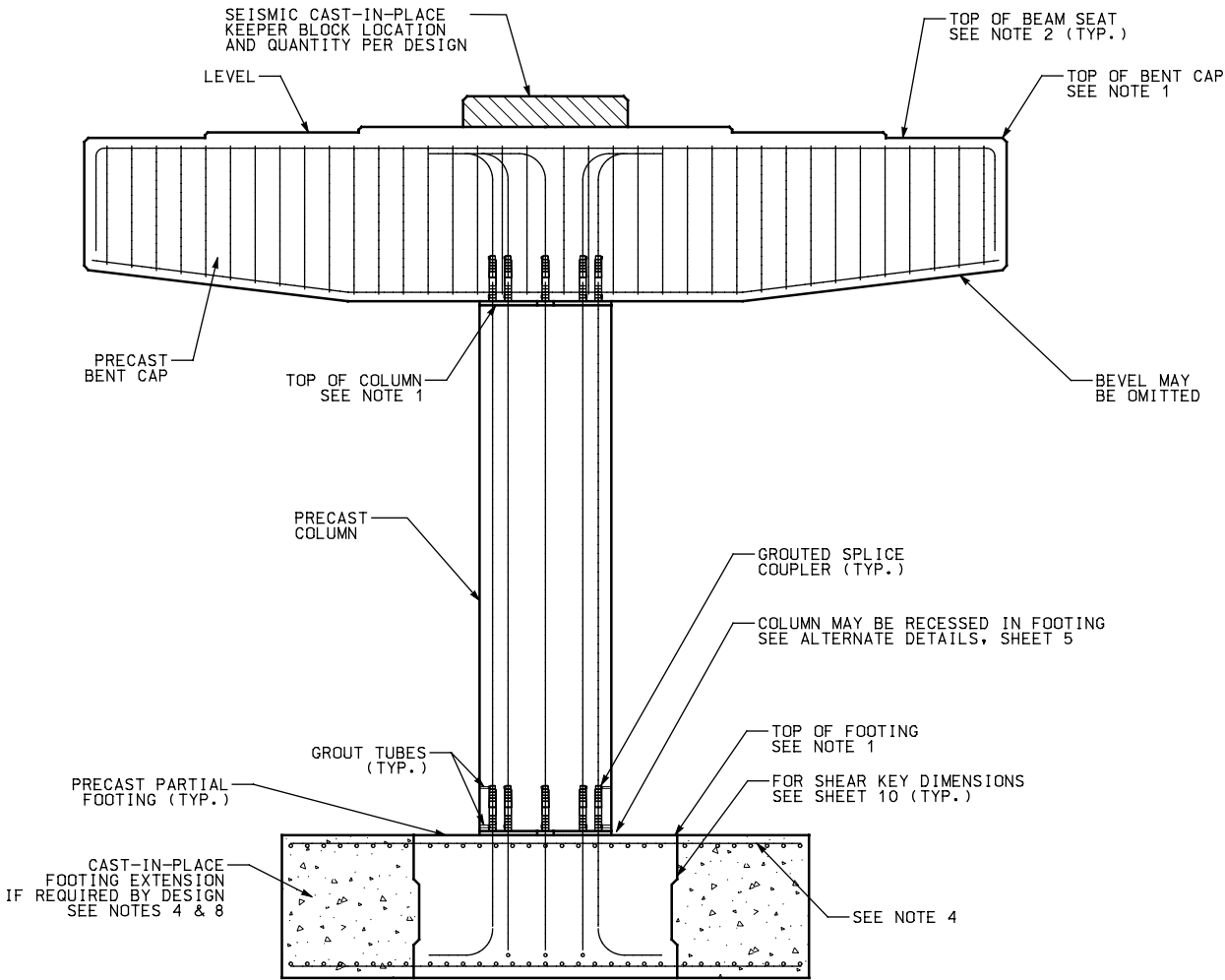
Guidelines For Accelerated Bridge Construction

10. NCHRP 12-69 Final Project Report, *"Guidelines For Design And Construction Of Decked Precast, Prestressed Concrete Girder Bridges."*
11. Wipf, Klaiber, and Hockerman, *Precast Concrete Elements for Accelerated Bridge Construction*, IHRB Project TR-561, Iowa Highway Research Board and the Iowa Department of Transportation, January 2009.
12. *Emulating Cast-in-Place Detailing in Precast Concrete Structure* ACI 550.1R-01, American Concrete Institute, Farmington Hills, MI, 2001.
13. *Building Code Requirements for Structural Concrete (ACI 318-08)*, American Concrete Institute Committee 318, 2008.
14. Scholz, Wallenfelsz, Cintia, Roberts-Wollmann, *"Recommendations for the Connection Between Full Depth Precast Bridge Deck Panel Systems and Precast I-Beams,"* Final Report Number VTRC 07-CR17, Virginia Transportation Research Council, Charlottesville, VA.
15. *Post-Tensioning Tendon Installation and Grouting Manual*, U.S. Department of Transportation, Federal Highway Administration, FHWA-NHI-13-026, May 2013.
16. *Specification for Grouting of PT Structures, Publication Number PTI M55.1-12*, Post Tensioning Institute, Farmington Hills, MI.
17. *Guide Specification for Grouted Post-Tensioning Publication Number PTI/ASBI M50.3-12*, Post Tensioning Institute, Farmington Hills, MI, and The American Segmental Bridge Institute.
18. *Acceptance Standards for Post-Tensioning Systems, PTI M50.1-98*, Post Tensioning Institute, Farmington Hills, MI.
19. ASBI, *Construction Practice Handbook For Segmental Concrete Bridges*, American Segmental Bridge Institute.
20. *Tests on Re-Bar Splices in Reinforced Concrete Columns using NMB Splice Sleeves*, Splice Sleeve North America (AKA: Splice Sleeve Japan LTD.), Report Number NPD-024.

PRECAST CONCRETE SUBSTRUCTURES NOTES																	
<div>1</div> <div>GUIDELINES</div> <div>THESE GUIDELINE DRAWINGS REPRESENT TYPICAL DETAILS FOR THE DESIGN AND DETAILING OF PRECAST CONCRETE SUBSTRUCTURES.</div> <div>THESE SHEETS ARE INCLUDED TO PROVIDE AN EXAMPLE OF THE DRAFTING LAYOUT OF TYPICAL PRECAST CONCRETE SUBSTRUCTURES. SEVERAL DIFFERENT SUBSTRUCTURE TYPES ARE SHOWN. THE DETAILS COVER A MAJORITY OF THE SUBSTRUCTURES USED IN THE NORTHEAST.</div> <div>DETAILS AND REINFORCEMENT SHOWN ARE SCHEMATIC. DESIGN AND DETAIL EACH SUBSTRUCTURE ACCORDING TO THE SPECIFIC REQUIREMENTS OF EACH BRIDGE.</div> <div>SET THE LOCATION OF THE LONGITUDINAL REINFORCING STEEL BASED ON THE SIZE OF THE GROUTED SPLICE COUPLER, ANY REINFORCING STEEL AROUND THE COUPLER, AND THE CLEAR COVER REQUIREMENTS FOR THE ELEMENT. ACCOUNT FOR THIS IN THE DESIGN OF THE ELEMENT.</div> <div>RECOMMENDED MAXIMUM SIZES OF ELEMENTS: WIDTH: THE MAXIMUM WIDTH OF THE ELEMENT INCLUDING ANY PROJECTING REINFORCING SHOULD BE KEPT BELOW 12FT FOR SHIPPING REASONS WEIGHT: THE MAXIMUM WEIGHT OF EACH ELEMENT SHOULD BE KEPT TO LESS THAN 100KIP. HEIGHT: THE MAXIMUM HEIGHT OF ANY ELEMENT INCLUDING PROJECTING REINFORCING SHOULD BE KEPT TO LESS THAN 10FT FOR SHIPPING REASONS. LENGTH: THE MAXIMUM LENGTH OF ANY ELEMENT SHOULD BE KEPT TO LESS THAN 120 FEET</div>			<div>4</div> <div>GENERAL NOTES</div> <div>DESIGN PRECAST CONCRETE SUBSTRUCTURE ELEMENTS IN ACCORDANCE WITH THE LATEST EDITION OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS EXCEPT AS NOTED OTHERWISE.</div> <div>PROVIDE CLEAR COVER FOR REINFORCING AND GROUTED SPLICE COUPLERS AS PER STATE STANDARDS IN ALL SUBSTRUCTURE ELEMENTS UNLESS OTHERWISE NOTED.</div> <div>CONTRACTOR TO PROVIDE TEMPORARY BRACING FOR ALL ELEMENTS UNTIL CONNECTIONS HAVE ACHIEVED ADEQUATE STRENGTH.</div> <div>THE CONTRACTOR MAY SUBSTITUTE ALTERNATE LEVELING DEVICES PROVIDED THEY CAN PRODUCE A STRUCTURE WITHIN THE SPECIFIED ERECTION TOLERANCES.</div> <div>CHAMFER ALL EXPOSED EDGES AND CORNERS 3⁄4".</div> <div>SHOW ESTIMATED SHIPPING WEIGHTS FOR ALL PRECAST ELEMENTS ON CONTRACT DRAWINGS.</div> <div>PROVIDE MILD REINFORCEMENT AS PER STATE SPECIFICATIONS UNLESS OTHERWISE NOTED.</div>			<div>6</div> <div>CONCRETE NOTES</div> <div>PRECAST CONCRETE: IN GENERAL, DESIGNERS SHOULD SPECIFY CONCRETE WITH A MINIMUM COMPRESSIVE STRENGTH OF 5000 PSI. THE MIX DESIGN OF THE PRECAST CONCRETE SHOULD NORMALLY BE DEVELOPED BY THE PRECAST FABRICATOR AND APPROVED BY THE OWNER.</div> <div>SITE CAST CONCRETE AND GROUT: THE DESIGNER SHALL SPECIFY THE MINIMUM CONCRETE PROPERTIES FOR THE FINAL CONSTRUCTION (STRENGTH, CURE TIME, ETC.). THE ENGINEER RESPONSIBLE FOR THE ASSEMBLY PLAN SHALL SPECIFY THE REQUIRED CONCRETE STRENGTHS FOR VARIOUS STAGES OF THE ASSEMBLY BASED ON CALCULATIONS DEVELOPED FOR THE ASSEMBLY PLAN. FOR EXAMPLE: THE ASSEMBLY PLANS COULD SPECIFY A CONCRETE STRENGTH IN A CLOSURE POUR OF 2000 PSI FOR A CERTAIN STAGE OF CONSTRUCTION, PROVIDED THAT THE CONCRETE GAINS THE FULL DESIGN STRENGTH PRIOR TO OPENING THE BRIDGE TO TRAFFIC.</div> <div>RECOMMENDATIONS FOR SITE CAST CONCRETE CONCRETE MIXES: MOST STATES HAVE STANDARD CONCRETE MIXES FOR BRIDGE CONSTRUCTION USING CONVENTIONAL CONSTRUCTION. ACCELERATED BRIDGE CONSTRUCTION PROJECTS OFTEN REQUIRE CONCRETE THAT CAN GAIN STRENGTH AND CURE IN A RAPID MANNER. MATERIAL PERFORMANCE SPECIFICATIONS ARE RECOMMENDED IN LIEU OF RIGID PRESCRIPTIVE SPECIFICATIONS. THE FOLLOWING CONCRETE STRENGTH PARAMETERS ARE SUGGESTED FOR USE ON PREFABRICATED BRIDGE PROJECTS. VERY EARLY STRENGTH CONCRETE: CONCRETE THAT WILL ATTAIN THE DESIGN STRENGTH IN LESS THAN 12 HOURS EARLY STRENGTH CONCRETE: CONCRETE THAT WILL GAIN THE DESIGN STRENGTH IN LESS THAN 24 HOURS NORMAL CONCRETE: CONCRETE THAT WILL GAIN THE DESIGN STRENGTH IN LESS THAN 7 DAYS SHRINKAGE OF EARLY STRENGTH CONCRETE CAN LEAD TO CRACKING. FOR THIS REASON, SHRINKAGE COMPENSATING ADMIXTURES SHOULD BE CONSIDERED. LIQUID ADMIXTURES SHOULD BE USED IN LIEU OF EXPANSIVE METALLIC POWDERS.</div> <div>IT IS RECOMMENDED THAT THE STATES WORK WITH LOCAL READY MIX PRODUCERS TO DEVELOP ACCEPTABLE MIX DESIGNS THAT CAN MEET THE REQUIRED PARAMETERS. IDEALLY, THESE MIXES SHOULD BE DEVELOPED PRIOR TO BIDDING AN ACCELERATED BRIDGE CONSTRUCTION PROJECT.</div> <div>CONTROLLED DENSITY FILL (FLOWABLE FILL): CONTROLLED DENSITY FILL CAN BE USED TO FILL VOIDS THAT ARE NOT SUBJECTED TO HIGH UNIT STRESSES AND ARE NOT REINFORCED. CONTROLLED DENSITY FILLS ARE FLOWABLE AND ARE LESS EXPENSIVE THAN FLOWABLE GROUTS. THIS WILL NORMALLY INCLUDE AREAS THAT ARE USED TO SEAT FOOTINGS AND SLABS. TYPICAL AREAS INCLUDE VOIDS UNDER FOOTINGS AND APPROACH SLABS. CONTROLLED DENSITY FILLS HAVE RELATIVELY SLOW SET TIMES. USE GROUT TO FILL VOIDS IF FAST SET TIMES ARE REQUIRED.</div> <div>GROUT: GROUT SHOULD ONLY BE USED FOR SMALL VOID GROUTING. THE REQUIRED STRENGTH OF THE GROUT SHOULD BE DETERMINED AND SPECIFIED BY THE DESIGN ENGINEER. NORMALLY THE DESIGN STRENGTH IS THE SAME STRENGTH AS THE SURROUNDING CONCRETE.</div> <div>FLOWABLE GROUT SHOULD BE SPECIFIED IN AREAS THAT REQUIRE SIGNIFICANT HORIZONTAL FLOW OF THE GROUT IN ORDER TO FILL THE VOID. THIS WOULD NORMALLY INCLUDE BEAM HAUNCHES AND HORIZONTAL JOINTS BETWEEN VERTICAL ELEMENTS.</div> <div>FOR COMPLEX VOIDS, THE ENGINEER MAY SPECIFY A TEST MOCK-UP GROUT POUR PRIOR TO THE ACTUAL CONSTRUCTION. THE MOCK-UP SHOULD BE SIMILAR TO THE FINAL CONFIGURATION. THE CONTRACTOR SHOULD BE REQUIRED TO DEMONSTRATE THAT THE GROUT CAN BE PLACED WITHOUT VOIDS. THIS CAN BE PROVEN BY DISMANTLING OF THE MOCK-UP AFTER GROUT CURING.</div>											
<div>2</div> <div>IMPLEMENTATION</div> <div>IT IS THE DESIGNER’S RESPONSIBILITY TO:</div> <div>DESIGN AND DETAIL ALL SUBSTRUCTURE ELEMENTS, INCLUDING BUT NOT LIMITED TO, COMPONENTS SUCH AS PIERS, ABUTMENTS, FOOTINGS AND FOUNDATIONS.</div> <div>DESIGN AND CHECK THE SUBSTRUCTURE ELEMENTS FOR ALL ANTICIPATED LOADS.</div> <div>DETAIL DIMENSIONS OF ALL ELEMENTS INCLUDING INTERNAL REINFORCING.</div> <div>SPECIFY AND DETAIL TOLERANCES FOR BOTH FABRICATION AND INSTALLATION OF ALL ELEMENTS. SEE TOLERANCE NOTES AND DETAILS.</div> <div>CALCULATE ELEVATIONS OF TOP OF ALL PRECAST ELEMENTS. ELEVATIONS TO BE INCLUDED ON ALL DETAILS.</div> <div>DETERMINE THE GEOTECHNICAL REQUIREMENTS OF THE SITE AND PLACE THE APPLICABLE INFORMATION ON THE PLANS.</div> <div>PLACE APPLICABLE GENERAL NOTES ON THE PLAN SET.</div> <div>ENSURE SUFFICIENT DETAIL IS ADDED TO THE DESIGN PLANS TO ENSURE PROPER FIT UP OF PRECAST ELEMENTS IN THE FIELD. TOLERANCE DETAIL SHEETS DEPICT A WORKING METHOD FOR ACHIEVING PROPER FIT UP.</div>			<div>5</div> <div>TOLERANCES</div> <div>ALL PRECAST CONCRETE ELEMENTS TO BE FABRICATED TO THE SPECIFIED DIMENSIONS WITHIN ACCEPTABLE INDUSTRY TOLERANCES. THE DETAILING AND LAYOUT OF PRECAST ELEMENTS SHOULD ACCOUNT FOR THE FABRICATION AND ERECTION TOLERANCES.</div> <div>THE DESIGNER SHOULD SPECIFY AND DETAIL ELEMENT FABRICATION TOLERANCES, ELEMENT ERECTION AND INSTALLATION TOLERANCES (BOTH HORIZONTAL AND VERTICAL), AND PILE DRIVING TOLERANCES (IF APPLICABLE).</div> <div>RECOMMENDED ELEMENT FABRICATION TOLERANCES ARE SHOWN ON SHEETS 11 AND 12. THESE ARE BASED ON INDUSTRY PRACTICE AND SHOULD ONLY BE REDUCED AFTER CONSULTATION WITH FABRICATORS. IF PRECAST ELEMENTS ARE TO BE CONNECTED TO CAST-IN-PLACE CONCRETE, COORDINATE TOLERANCES BETWEEN SHOP AND FIELD PERSONNEL.</div> <div>RECOMMENDED ELEMENT ERECTION TOLERANCES ARE SHOWN ON VARIOUS DETAILS WITHIN THESE GUIDE DETAILS. HORIZONTAL ERECTION TOLERANCES ARE ALWAYS BASED ON MEASUREMENTS FROM A COMMON WORKING POINT OR LINE. ERECTION OF ELEMENTS BASED ON CENTER TO CENTER SPACING SHOULD NOT BE USED AS THIS COULD LEAD TO BUILD UP OF ERECTION ERRORS.</div> <div>THE WIDTH OF JOINTS BETWEEN ELEMENTS ARE A FUNCTION OF ELEMENT TOLERANCES, ERECTION TOLERANCES, AND PLACEMENT OF FILL MATERIALS. THE WIDTH OF JOINTS SHOWN IN THESE GUIDE DETAILS SHOULD NOT BE REDUCED WITHOUT CAREFUL CONSIDERATION OF TOLERANCES.</div> <div>VERTICAL ERECTION TOLERANCES SHOULD BE MEASURED DURING ERECTION AT THE TOP OF EACH ELEMENT AS SHOWN ON THE GUIDE DETAILS. HORIZONTAL JOINTS ARE PROVIDED TO ACCOMMODATE ELEMENT HEIGHT TOLERANCES DURING ERECTION.</div> <div>SEVERAL OF THE GUIDE DETAILS MAKE USE OF GROUTED SPLICE COUPLERS. CONTRACTORS MAY CHOOSE TO EXTEND REINFORCING BARS FROM ADJACENT ELEMENTS WITH EXTRA LENGTH. THE BAR EXTENSIONS CAN THEN BE CUT TO THE COUPLER MANUFACTURER’S RECOMMENDED LENGTH BASED ON REQUIRED ELEMENT ERECTION TOLERANCES.</div> <div>DETAILS FOR FOUNDATION VOIDS ARE BASED ON A LATERAL INSTALLATION TOLERANCE OF ±3 INCHES IN PLAN FOR DRILLED SHAFTS AND DRIVEN PILES. IF THIS CANNOT BE ACCOMPLISHED THE DETAILS SHOWN HEREIN SHOULD BE ADJUSTED IN ORDER TO ACCOMMODATE THE ANTICIPATED TOLERANCES. FOR DETERMINING THE MINIMUM SIZE OF THE CORRUGATED METAL PIPE FOR USE WITH THE SUBSTRUCTURE ELEMENTS USE THE FOLLOWING: PIPE PILE: O.D. + 8" H-PILE OR PRECAST PILE: PILE DIA. + 8" (MEASURED DIAGONALLY FROM CORNERS) DRILLED SHAFT: O.D. OF SPIRAL + 8" THIS IS BASED ON PROVIDING A 1" MINIMUM GAP BETWEEN THE PILE AND THE SIDE OF THE VOID WITH THE MAXIMUM OFFSET TOLERANCE. NOTE THAT CORRUGATED METAL PIPE SIZES ARE BASED ON INSIDE DIAMETER.</div>			<div>7</div> <div>INDEX OF SHEETS</div> <div>1 PRECAST CONCRETE SUBSTRUCTURE NOTES 2 SINGLE COLUMN BENT 3 MULTI-COLUMN BENT 4 WALL PIER 5 COLUMN DETAILS 6 TYPICAL PIER CONNECTION DETAILS 7 FOOTING DETAILS 8 INTEGRAL ABUTMENT DETAILS 9 CANTILEVER ABUTMENT DETAILS 10 MISCELLANEOUS ABUTMENT DETAILS 11 PIER ELEMENTS TOLERANCES 12 ABUTMENT AND WALL ELEMENT TOLERANCES</div>											
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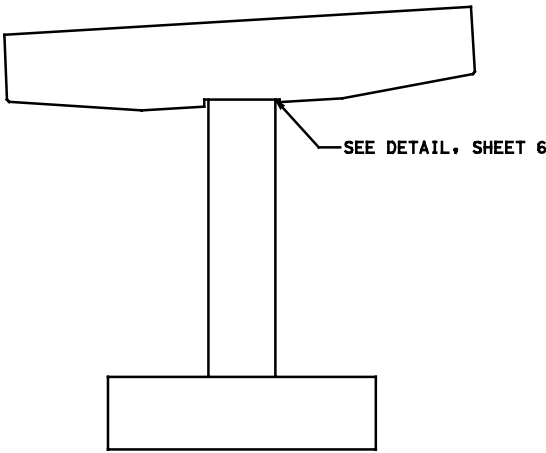


① PRECAST SINGLE COLUMN BENT
SIDE VIEW



② PRECAST SINGLE COLUMN BENT
ELEVATION

NOTE: PIER CAP TOP CAN BE MODIFIED
TO STATE STANDARDS.



③ SCHEMATIC SLOPED
PIER CAP DETAIL

④ NOTES

- ERECTION TOLERANCE ON ELEVATION $\pm 1/4"$.
- ERECTION TOLERANCE ON BEAM SEAT ELEVATION $\pm 1/2"$. MAY BE SET HIGH AND GROUND TO SPECIFIED ELEVATION.
- COLUMN SHEAR REINFORCEMENT NOT SHOWN FOR CLARITY.
- DETAIL BAR EXTENSIONS TO THE LIMITS OF THE FOOTING IF POSSIBLE. IF TOTAL WIDTH OF FOOTING AND BAR EXTENSIONS EXCEEDS SHIPPING LIMITS, THEN DETAIL AS LAP SPLICES IN REINFORCING OR ADD MECHANICAL BAR SPLICERS (IF REQUIRED).
- A LEVEL PIER CAP IS PREFERRED TO REDUCE FABRICATION COSTS. SLOPED CAPS ARE PERMITTED AS SHOWN IN THE SCHEMATIC DETAIL.
- SEISMIC KEEPER BLOCK MAY BE PLACED BETWEEN OTHER BEAMS IF REINFORCING CONFLICTS ARISE.
- FOOTING TO BE SET TO A TOLERANCE OF $\pm 1/4"$ IN 4 FEET.
- USE CAST IN PLACE EXTENSIONS TO KEEP SIZE AND WEIGHT OF FOOTING WITHIN RECOMMENDED MAXIMUM SIZE LIMITS.

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PRECAST/PRESSED CONCRETE INSTITUTE NORTHEAST

SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES

SINGLE COLUMN BENT

DISCLAIMER: The details shown are guidelines and should not be considered standards. The information has been obtained from sources believed to be reliable. PCI or its members shall not be responsible for any errors, omissions or damages arising out of this information. PCI has published this work with the understanding that PCI is supplying information only. PCI is not rendering engineering or other professional services through this guideline. If such services are required, please seek an appropriate professional.

ISSUE DATE: 06-05-12

SHEET: SUB-2

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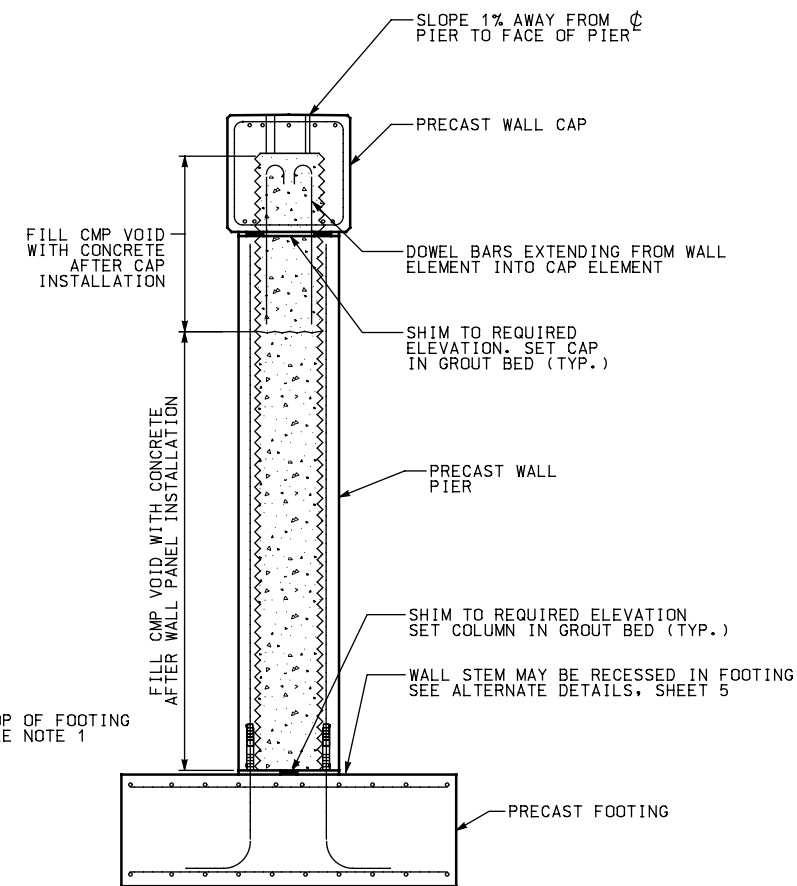
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| <p>SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES</p> <p>MULTI-COLUMN BENT</p> <p>DISCLAIMER: The details shown are guidelines and should not be considered standards. The information has been obtained from sources believed to be reliable. Certain limitations apply to any entity, organization or individual who uses the information. PCI is not responsible for any damages or liabilities arising out of the information. PCI has provided this information with the understanding that PCI is not providing engineering or other professional services through this guideline. If such services are required, please seek an appropriate professional.</p> | | <p>ISSUE DATE: 06-05-12</p> <p>SHEET: SUB-3</p> | |

④ NOTES

- SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES**
- WALL PIER**
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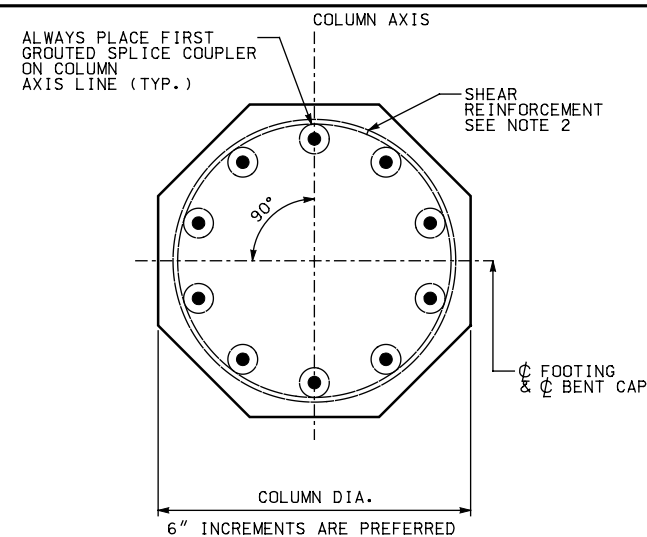


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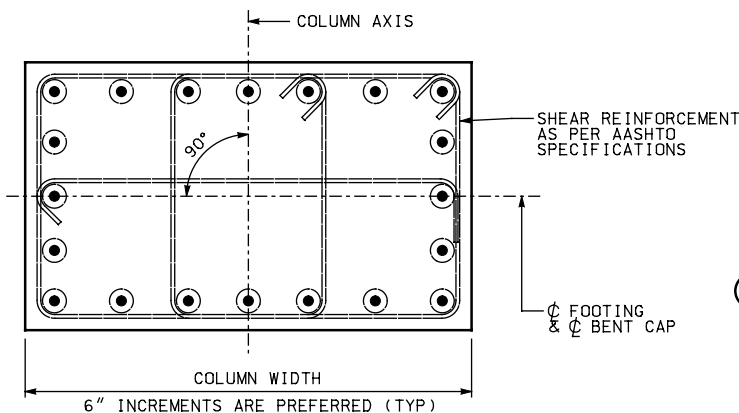
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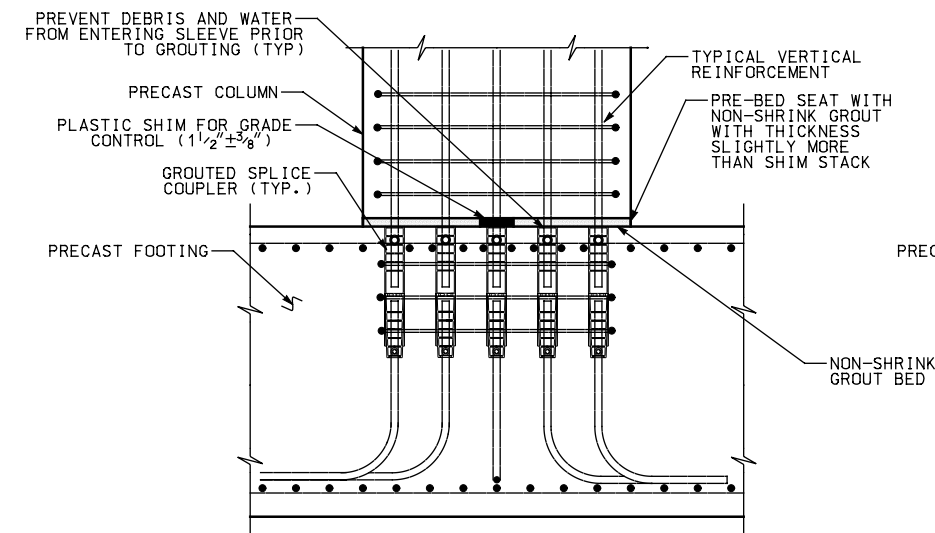
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1 TYPICAL OCTAGONAL COLUMN SECTION

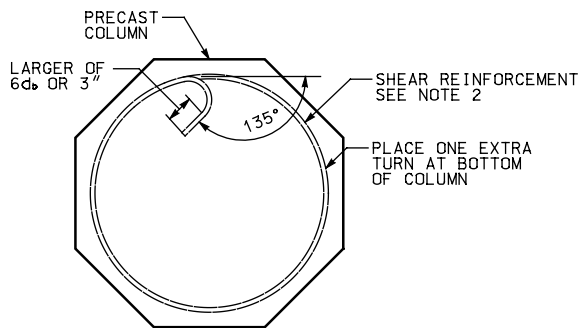


2 TYPICAL RECTANGULAR COLUMN SECTION



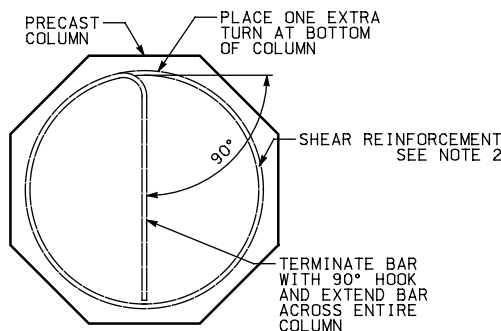
5 OPTIONAL PRECAST COLUMN TO FOOTING CONNECTION

NOTE: THE GROUTED SPLICE COUPLERS MAY BE INSTALLED IN THE PRECAST COLUMN, RATHER THAN THE FOOTING.



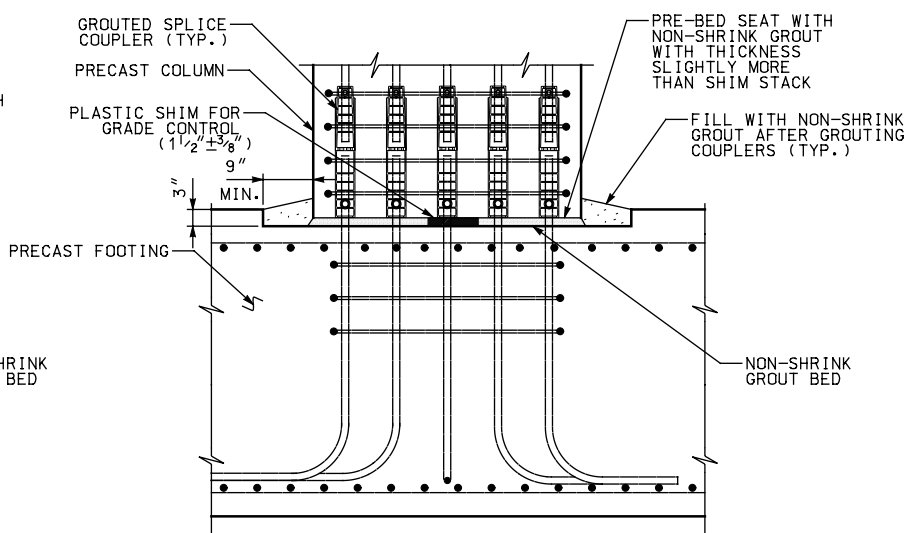
3 SHEAR REINFORCEMENT SPIRAL REINFORCEMENT TERMINATION PER AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

NOTE: COLUMN VERTICAL REINFORCEMENT NOT SHOWN FOR CLARITY.



4 SHEAR REINFORCEMENT SPIRAL REINFORCEMENT TERMINATION PER AASHTO SEISMIC GUIDE SPECIFICATIONS

NOTE: COLUMN VERTICAL REINFORCEMENT NOT SHOWN FOR CLARITY.



6 OPTIONAL RECESSED FOOTING - PREFERRED

NOTE: COLUMN SHOWN, WALL PANEL SIMILAR.

7 COLUMN NOTES

1. OCTAGONAL CROSS SECTIONS ARE PREFERRED DUE TO EASE OF FABRICATION. OTHER SECTIONS ARE ALLOWED.
2. SHEAR REINFORCEMENT USED FOR TRANSVERSE COLUMN CONFINEMENT REINFORCEMENT CONSISTS OF SPIRALS OR HOOPS.
3. IT IS RECOMMENDED TO PLACE THE FIRST GROUTED SPLICE COUPLER ON THE COLUMN AXIS LINE TO FACILITATE EASE OF CONSTRUCTION.
4. SOME GROUTED SPLICE COUPLER MANUFACTURERS ALLOW THE USE OF OVERSIZE COUPLERS IN ORDER TO INCREASE THE SETTING TOLERANCES FOR ELEMENTS. THIS SHOULD ONLY BE ALLOWED IF SUPPORTED BY TEST DATA.

GROUTED SPLICE COUPLER DIMENSIONS

BAR SIZE	OUTSIDE DIAMETER (IN.)	LENGTH OF COUPLER (IN.)
4	2.625	14.125
5	3.000	14.125
6	3.000	14.125
7	3.000	18.75
8	3.500	18.75
9	3.500	18.75
10	3.500	23.5
11	4.000	23.5
14	4.000	28.375
18	4.500	39.625

USE THIS TABLE FOR DETAILING OF ELEMENT REINFORCEMENT INCLUDING SPACING, COVER, AND EMBEDMENT LENGTHS. IN MOST CASES, THESE DIMENSIONS WILL WORK FOR OVERSIZED COUPLERS. IF THE FABRICATOR ELECTS TO OVERSIZE A COUPLER, THESE REQUIREMENTS SHALL BE CHECKED DURING THE DEVELOPMENT OF SHOP DRAWINGS.

SOURCES: MATERIAL SPECIFICATIONS FROM THE THREE MOST COMMON SUPPLIERS (NMB SPLICE SLEEVE, LENTON-ERICO, DAYTON SUPERIOR)

8 GROUTED SPLICE COUPLER CONNECTION SEQUENCE

1. IT IS RECOMMENDED THAT THE GROUTING PROCEDURE BE COMPLETED IN THE PRESENCE OF A CONTRACTOR'S SUPERVISOR THAT IS EXPERIENCED IN THE INSTALLATION OF GROUTED SLEEVES. MANUFACTURER TRAINING MAY BE REQUIRED FOR INEXPERIENCED STAFF.
2. FOLLOW THE WRITTEN INSTALLATION PROCEDURES OF THE COUPLER MANUFACTURER. THE FOLLOWING ARE GENERAL PROCEDURES THAT APPLY TO MOST COUPLER MANUFACTURERS.
3. IT IS RECOMMENDED THAT THE ELEMENT WITH THE REINFORCEMENT BAR EXTENSIONS BE FABRICATED WITH EXTENDED LENGTHS.
4. SURVEY LOCATION AND ELEVATION OF LOWER ELEMENT.
5. DETERMINE THE REQUIRED REINFORCING BAR EXTENSION LENGTHS AND THE REQUIRED SHIM HEIGHTS BASED ON THE SURVEY.
6. CUT THE BAR EXTENSIONS TO THE REQUIRED LENGTH BASED ON THE SURVEY AND THE COUPLER MANUFACTURER'S RECOMMENDATIONS. FOR COATED BARS, THE ENDS OF THE BARS NEED NOT BE RE-COATED.
7. PLACE BEDDING GROUT ON TOP OF LOWER ELEMENT. THE USE OF EXTRA GROUT THAT IS ALLOWED TO FLOW OUT DURING ELEMENT PLACEMENT IS RECOMMENDED. IN LIEU OF PRE-PLACEMENT OF BEDDING GROUT, THE BEDDING GROUT CAN BE FLOWED INTO PLACE AFTER ELEMENT ERECTION BUT PRIOR TO GROUTING OF COUPLERS.
8. ERECT UPPER ELEMENT TO WITHIN THE SPECIFIED ERECTION TOLERANCES. PREVENT BEDDING GROUT FROM FLOWING INTO COUPLER.
9. MAINTAIN INTEGRITY OF GROUT BED DURING SETTING OPERATION. REPAIR GROUT THAT IS DISPLACED OR GAPS THAT DEVELOP IN THE GROUT JOINT USING HAND TOOLS.
10. BRACE THE UPPER ELEMENT.
11. INSTALL GROUT IN COUPLERS FOLLOWING THE MANUFACTURER'S WRITTEN PROCEDURES. IF THE COUPLER IS BELOW THE JOINT, THE COUPLER GROUT CAN BE INSTALLED PRIOR TO APPLICATION OF BEDDING GROUT.
12. ERECTION OF SUBSEQUENT ELEMENTS ABOVE A CONNECTION SHOULD NOT COMMENCE UNTIL THE CONNECTION HAS ACHIEVED ADEQUATE STRENGTH AS DETERMINED THROUGH STRENGTH TESTING OF THE GROUT. THE TIMING OF SUBSEQUENT CONSTRUCTION STEPS SHOULD BE SPECIFIED IN THE BRIDGE ASSEMBLY PLAN.

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SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES

COLUMN DETAILS

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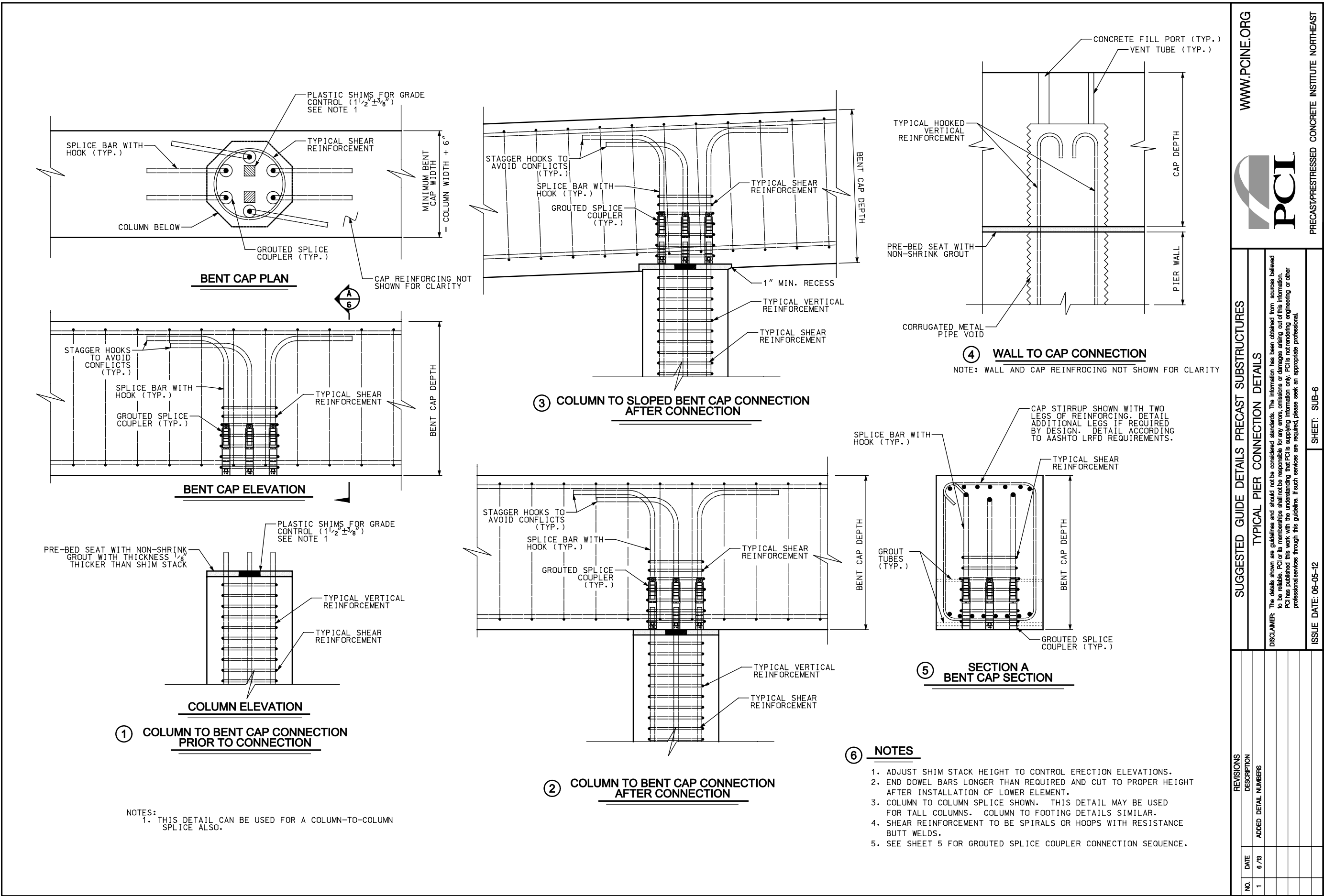
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⑤ PRECAST FOOTING ON BATTERED PILE

1. CONTRACTOR TO DETERMINE SIZE AND SPACING OF PORTS BASED ON MIX DESIGN AND FOOTING SIZE.
2. ERECTION TOLERANCE ON ELEVATION $\pm 4"$. THIS SHOULD BE SPECIFIED AND DETAILED ON THE CONTRACTOR'S ASSEMBLY PLAN.
3. DETAIL BAR EXTENSIONS TO THE LIMITS OF THE FOOTING IF POSSIBLE. IF TOTAL WIDTH OF FOOTING AND BAR EXTENSIONS EXCEEDS SHIPPING LIMITS, THEN DETAIL AS LAP SPLICES IN REINFORCING OR ADD MECHANICAL BAR SPLICERS.
4. PROVIDE 3" CLEAR COVER FOR BOTTOM MATS OF REINFORCING.
5. THE DESIGNER SHOULD DETAIL ALL PERTINENT FOOTING REINFORCING AND RESOLVE POTENTIAL CONFLICTS WITH PILE VOIDS.
6. USE CAST-IN-PLACE EXTENSIONS TO KEEP SIZE AND WEIGHT OF PRECAST FOOTING WITHIN THE RECOMMENDED MAXIMUM SIZE LIMITS.
7. PARTIAL PRECAST FOOTINGS MAY BE USED WITH PILES OR DRILLED SHAFTS.
8. PARTIAL PRECAST FOOTINGS MAY BE USED TO CONNECT ADJACENT FOOTINGS TO CREATE A CONTINUOUS FOOTING.
9. IN GENERAL, A PILE SHOULD NOT BE PLACED DIRECTLY BELOW THE COLUMNS ABOVE UNLESS ALL REINFORCING CONFLICTS CAN BE RESOLVED.
10. IN MOST CASES FLOWABLE FILL WILL BE ACCEPTABLE FOR SEATING SPREAD FOOTINGS. NON-SHRINK GROUT SHOULD ONLY BE USED WHERE FOOTING PRESSURES ARE EXCESSIVE OR WHERE FAST SET TIMES ARE REQUIRED.



LEVELING BOLT NOTES

-
- Diagram illustrating the components and assembly of a cast-in-place pile footing:
- CAST-IN-PLACE FOOTING
 - PRECAST COLUMN OR WALL ELEMENT
 - TEMPORARY SUPPORT DEVICE (STEEL MEMBER CAST INTO COLUMN OR WALL)
 - PROJECTING REINFORCING (HOOKED OR HEADED BARS)
 - STEEL SHIMS
 - SOIL BEARING PLATE
 - STEEL H-PILE OR CONCRETE PILE
 - BOTTOM MAT REINFORCING MAY BE PLACED BELOW THE TOP OF PILE. REFER TO STATE STANDARDS FOR CAST-IN-PLACE PILE FOOTINGS

NOTE: PILE FOOTING SHOWN, SPREAD FOOTING SIMILAR
USE THIS OPTION FOR FOOTINGS WITH CLOSELY SPACED PILES

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SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES

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③ INTEGRAL ABUTMENT NOTES:

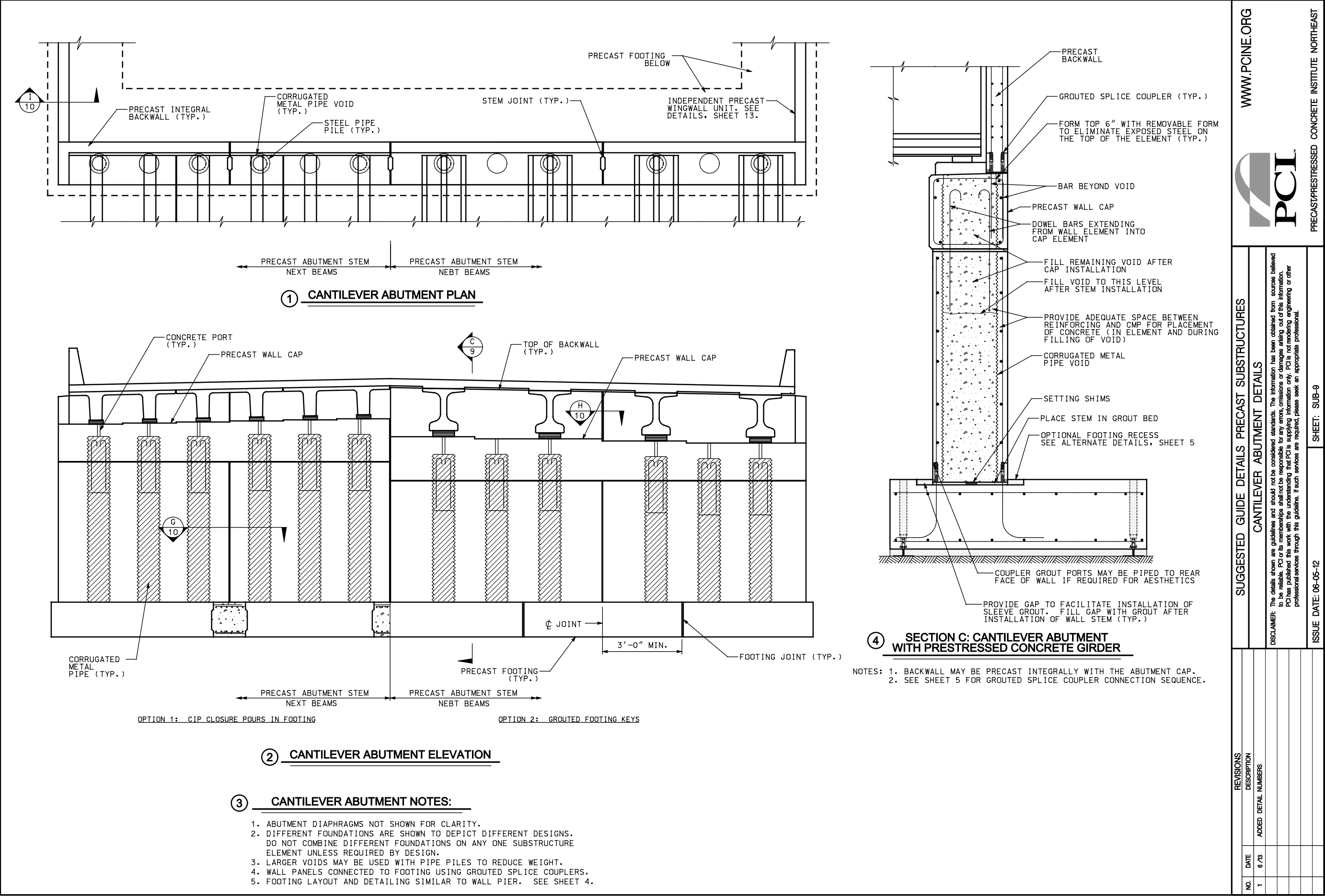
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- A detailed cross-sectional diagram of a pile cap and abutment stem assembly. The diagram shows a vertical pile cap at the top, which is part of a larger structure. Below the pile cap is a precast abutment stem. The space between the pile cap and the abutment stem is filled with concrete. A temporary erection pedestal is shown supporting the pile cap. A corrugated metal pipe void is located within the abutment stem. The diagram is labeled with various components and their connections.
- CAST-IN-PLACE CONCRETE INTEGRAL DIAPHRAGM
 - PRECAST BACKWALL
SEE NOTE 3
 - GROUTED SPLICE COUPLER
SEE NOTE 6 (TYP.)
 - TEMPORARY ERECTION PEDESTAL
 - GRADE
 - FILL VOID WITH CONCRETE
 - PRECAST ABUTMENT STEM
 - CORRUGATED METAL PIPE VOID
 - PILE OR DRILLED SHAFT

④ SECTION B: INTEGRAL ABUTMENT

-
- DECK
- APPROACH SLAB
- BACKWALL INTEGRAL WITH BEAM ENDS USING CAST-IN-PLACE CONCRETE
- GIRDER
- FORM TOP 6" WITH REMOVABLE FORM TO ELIMINATE EXPOSED STEEL ON THE TOP OF THE ELEMENT (TYP.)
- REINFORCING EXTENDED INTO BACKWALL POUR (TYP.)
- FOAM FILLER
- FILL VOID WITH CONCRETE
- PRECAST ABUTMENT STEM
- CORRUGATED METAL PIPE VOID
- GRADE
- PILE OR DRILLED SHAFT

(5) SECTION B: SEMI-INTEGRAL ABUTMENT

NOTE: BACKWALL CAN BE PRECAST INTEGRALLY WITH THE BEAM USING A SECONDARY POUR. SPECIAL DETAILING OF JOINTS BETWEEN BACKWALLS WILL BE REQUIRED.



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SUGGESTED GUIDE DETAILS PRECAST SUBSTRUCTURES
CANTILEVER ABUTMENT DETAILS

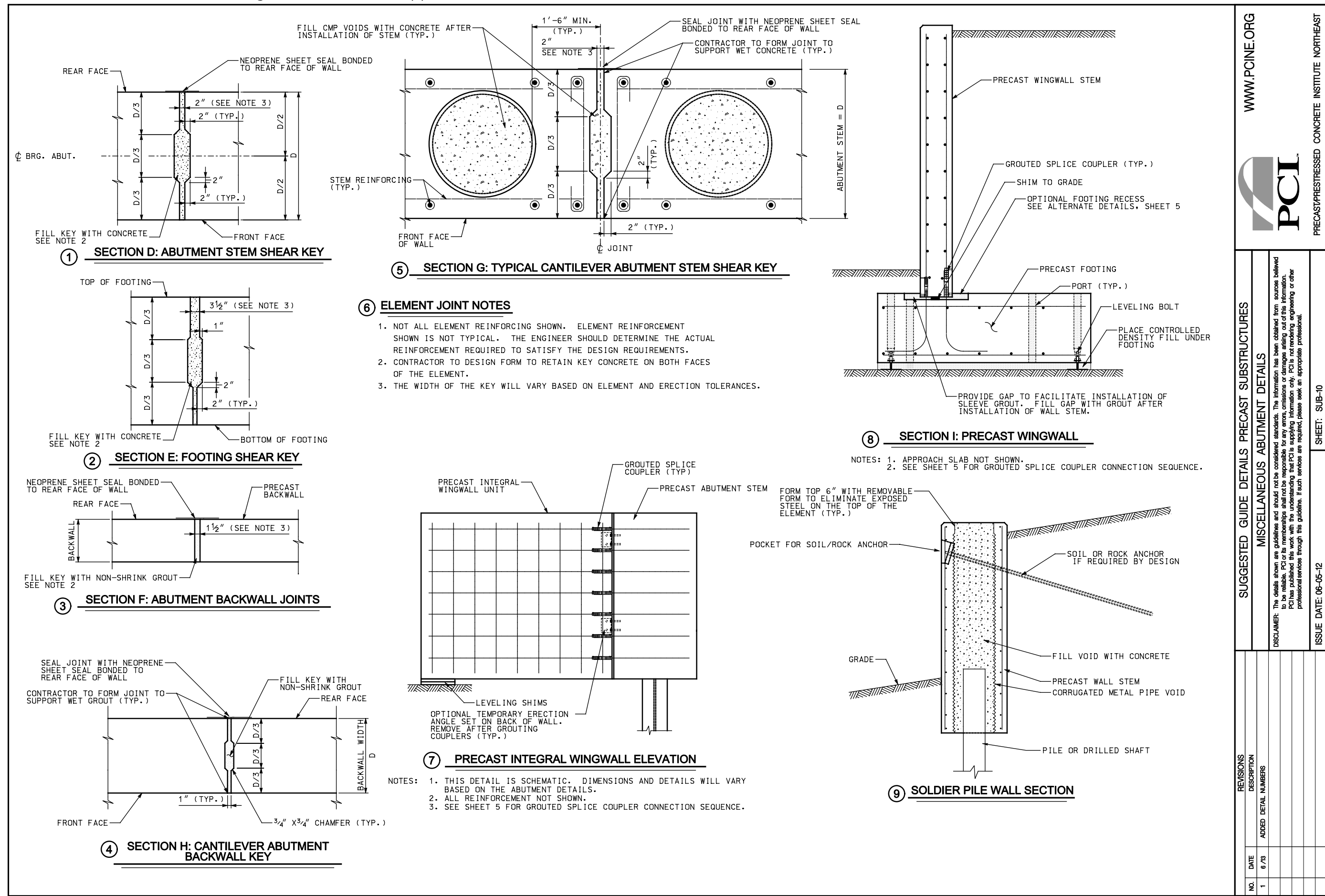
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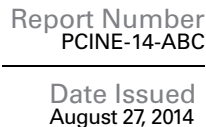
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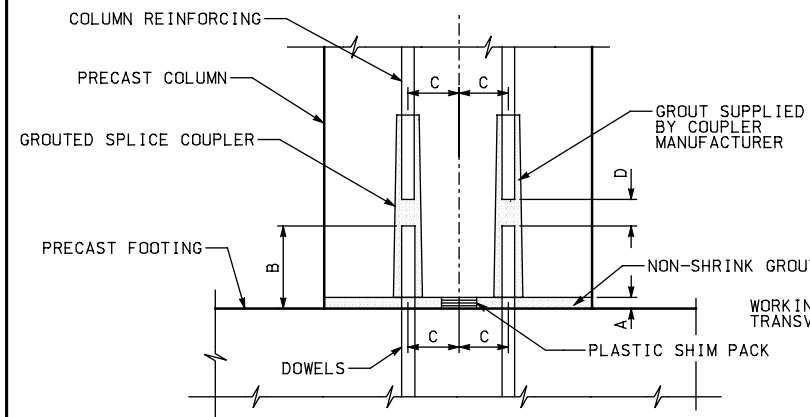
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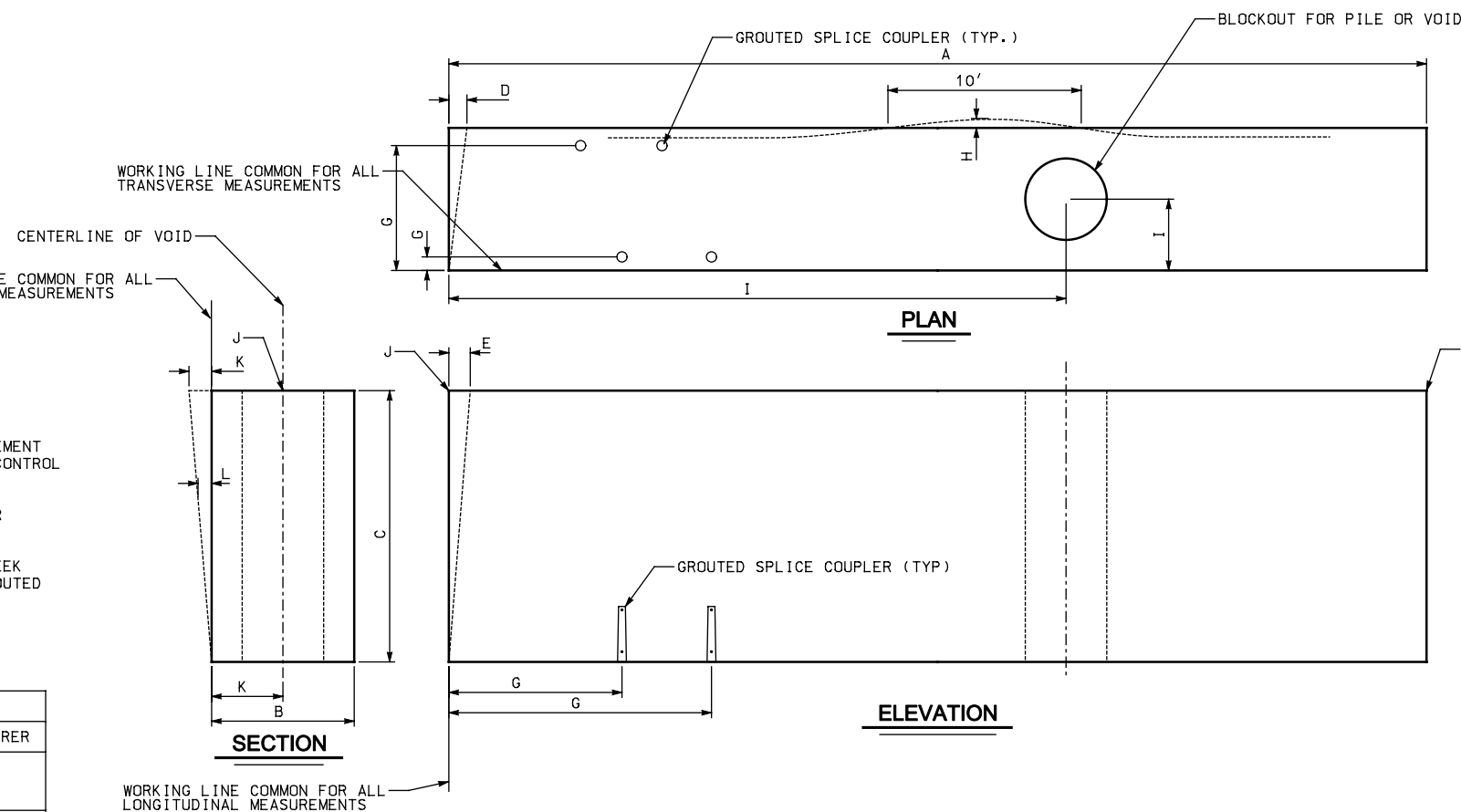


GROUTED SPLICE COUPLER DETAILS

- NOTES: 1. USE MATCHING TEMPLATES AND JIGS FOR THE LOCATION OF REINFORCEMENT AND GROUTED SPLICE COUPLER PLACEMENT WITHIN THE ELEMENTS TO CONTROL CRITICAL DIMENSION "C".
2. CONSULT MANUFACTURER OF THE GROUTED SPLICE COUPLER FOR PROPER DIMENSIONS "B" AND "D" AND FOR TOLERANCE ON THESE DIMENSIONS.
3. BEFORE EXECUTING GROUTED SPLICE COUPLER ASSEMBLIES, ALWAYS SEEK INSTALLATION RECOMMENDATIONS FROM THE MANUFACTURER OF THE GROUTED SPLICE COUPLER USED.

① GROUTED SPLICE COUPLER TOLERANCES

A	SHIM PACK HEIGHT	$1\frac{1}{2}'' \pm \frac{3}{8}''$
B	DOWEL HEIGHT	CONSULT MANUFACTURER
C	LOCATION OF COLUMN REINFORCING, GROUTED SPLICE COUPLER, AND FOOTING DOWELS MEASURED FROM A COMMON REFERENCE POINT	$\pm \frac{1}{4}''$
D	GAP BETWEEN DOWELS AND COLUMN REINFORCING	CONSULT MANUFACTURER



NOTE: THESE TOLERANCES APPLY TO THE FOLLOWING ELEMENTS
INTEGRAL ABUTMENT STEMS
CANTILEVER ABUTMENT STEMS
ABUTMENT BACKWALLS
CANTILEVER RETAINING WALLS

② WALL PANEL ERECTION TOLERANCES

J	TOP ELEVATION FROM NOMINAL TOP ELEVATION MAXIMUM LOW MAXIMUM HIGH	$\frac{1}{2}$ $\frac{1}{4}$
K	MAXIMUM PLUMB VARIATION OVER HEIGHT OF PANEL	$\frac{1}{2}$
L	PLUMB IN ANY 10 FEET OF PANEL HEIGHT	$\frac{1}{4}$ "

③ WALL PANEL FABRICATION TOLERANCES

A	LENGTH	$\pm 1/4''$
B	WIDTH (OVERALL)	$\pm 1/4''$
C	DEPTH (OVERALL)	$\pm 1/4''$
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	$\pm 1/8''$ PER 12 INCH WIDTH $\pm 1/2''$ MAXIMUM
E	VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	$\pm 1/8''$ PER 12 INCH WIDTH $\pm 1/2''$ MAXIMUM
G	LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	$\pm 1/4''$
H	LOCAL SMOOTHNESS OF ANY SURFACE	$\pm 1/4''$ IN 10 FEET
I	LOCATION OF BLOCKOUT FOR PILES OR VOIDS	$\pm 1''$

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ABUTMENT AND WALL ELEMENT TOLERANCES

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Diagram illustrating the cross-section of a bridge approach slab, showing two options for the longitudinal joint.

Labels and components:

- PRECAST BARRIER IN APPROACH SLAB PANEL
- OVERLAY (IF SPECIFIED BY AGENCY)
- APPROACH SLAB LONGITUDINAL JOINT, OPTION 1 (TYP.)
- APPROACH SLAB LONGITUDINAL JOINT, OPTION 2 (TYP.)
- WORKING LINE
- PLACE POLYETHYLENE SHEET TO FORM AGAINST SUBGRADE (TYP.)
- HORIZONTAL SETTING TOLERANCE $\pm 1/4"$

TYPICAL SECTION: SUB-SURFACE APPROACH SLAB WITH INTEGRAL BARRIER

BRIDGE PARAPET

EXPANSION JOINT

INTEGRAL APPROACH SLAB PARAPET

PRECAST APPROACH SLAB

PRECAST DECK PANEL

CAST-IN-PLACE CLOSURE POUR

GIRDER

BACKWALL

DISCONTINUE SLEEPER SLAB AT CURB TO FACILITATE APPROACH RAILING POST OR HIGHWAY GUARD RAIL TRANSITION ELEMENT INSTALLATION

PRECAST SLEEPER SLAB

STANDARD PARAPET END - SURFACE APPROACH SLAB
SKewed ISOMETRIC VIEW

NOTES: 1. CONNECTION TO APPROACH SLAB NOT SHOWN
2. SURFACE APPROACH SLAB SHOWN. SUB-SURFACE SLABS SIMILAR

OPTION 3: UNREINFORCED LONG

NOTES: 1. DO NOT USE THIS JOINT IN AN INTEGRAL BARRIER
2. DO NOT USE THIS JOINT WITH

2-# 5 (TYP.)

5 HEADED STUD REBAR CONNECTOR (TYP.)

11"

8"

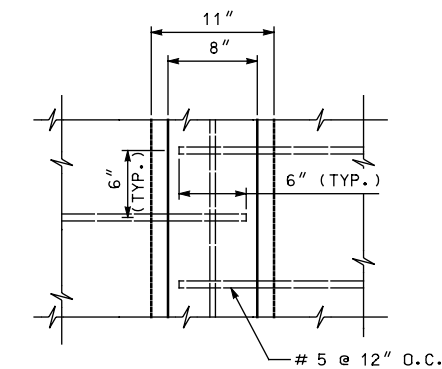
FILL JOINT WITH CONCRETE

8 1/2" TYP.

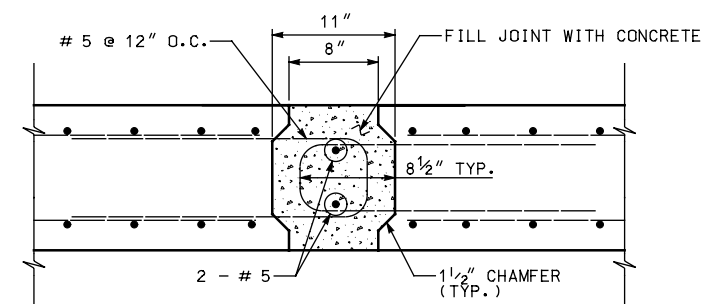
1 1/2" CHAMFER (TYP.)

**SECTION J: LONGITUDINAL JOINT
OPTION 1**

SECTION J: LONGITUDINAL JOINT OPTION 1

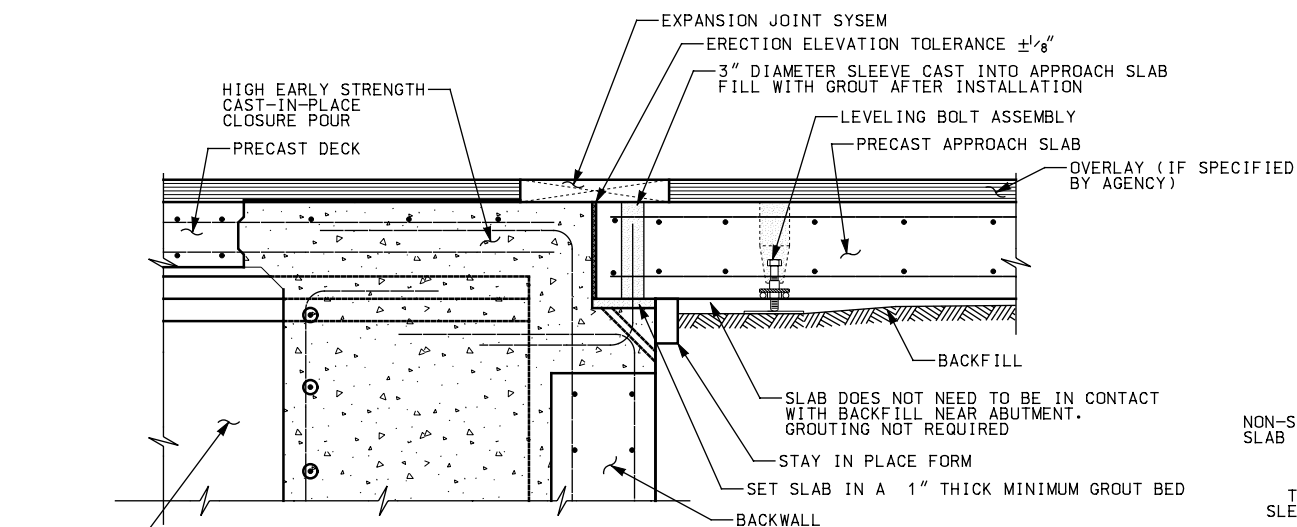


LONGITUDINAL JOINT PLAN
OPTION 2



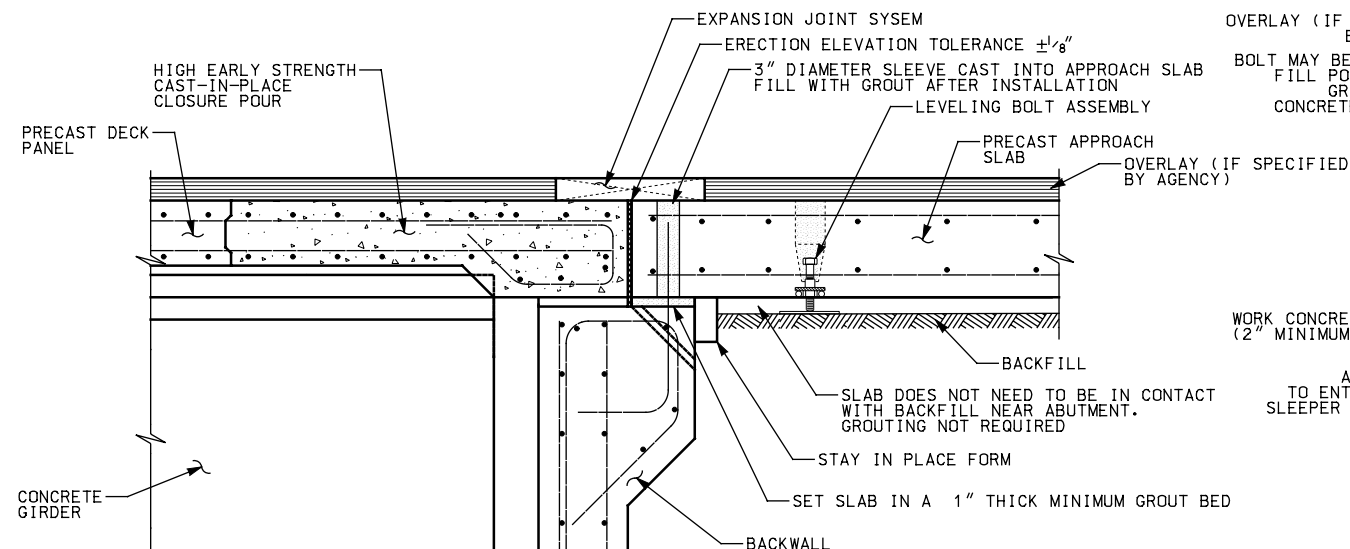
**SECTION K: LONGITUDINAL JOINT
OPTION 2**

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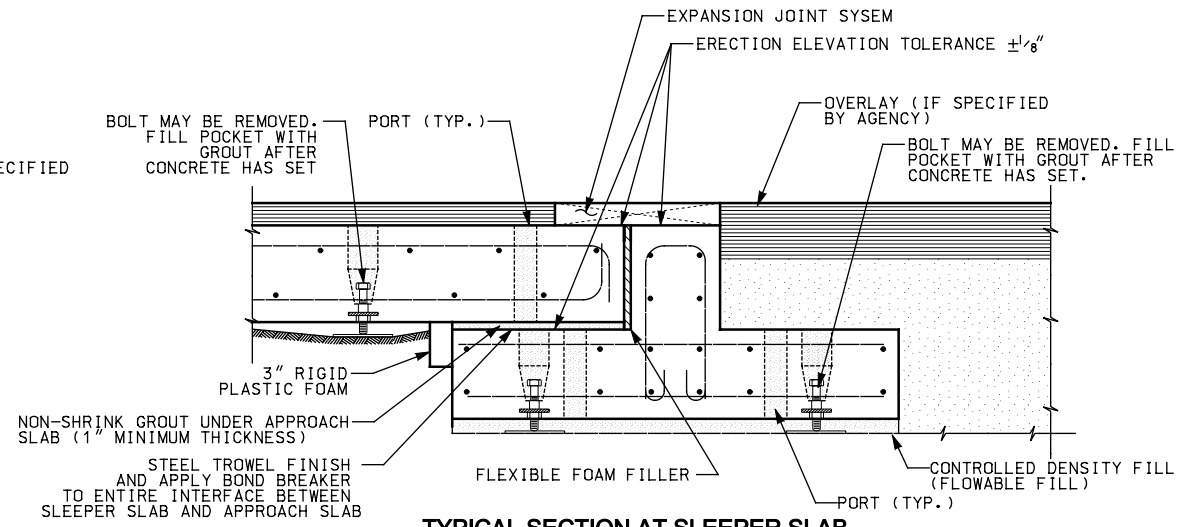
TYPICAL SECTION: SURFACE APPROACH SLAB AT INTEGRAL ABUTMENT

NOTES: CONCRETE GIRDER SHOWN STEEL BEAM SIMILAR.
SHIMS MAY BE USED IN LIEU OF LEVELING BOLTS (ALLOW ALTERNATES ON PLANS)



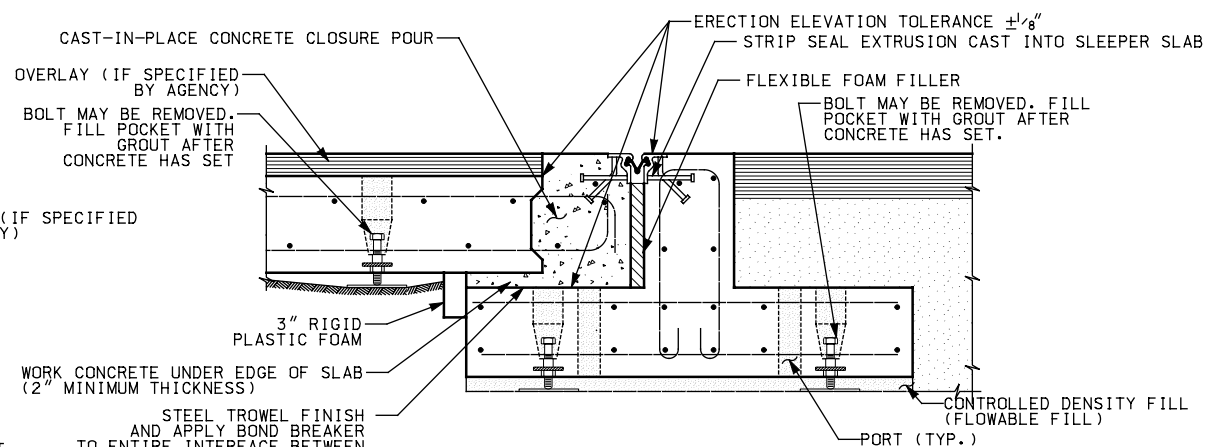
TYPICAL SECTION: SURFACE APPROACH SLAB AT CANTILEVER ABUTMENT

NOTES: CONCRETE GIRDER SHOWN STEEL BEAM SIMILAR.
SHIMS MAY BE USED IN LIEU OF LEVELING BOLTS (ALLOW ALTERNATES ON PLANS)



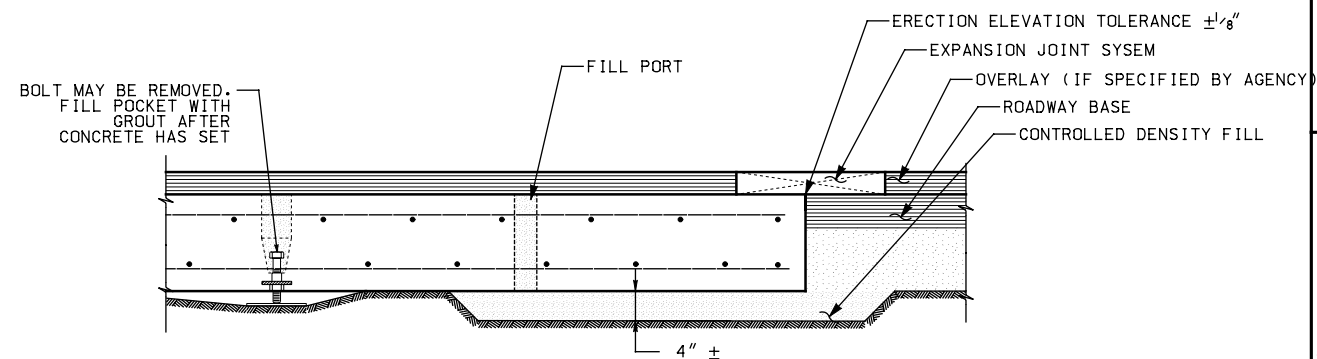
TYPICAL SECTION AT SLEEPER SLAB

NOTES: CONTRACTOR TO DETERMINE SIZE AND SPACING OF GROUT
PORTS BASED ON FLOWABLE FILL PROPERTIES
SHIMS MAY BE USED IN LIEU OF LEVELING BOLTS (ALLOW ALTERNATES ON PLANS)



**TYPICAL SECTION AT SLEEPER SLAB
STRIP EXPANSION JOINT**

NOTES: CONTRACTOR TO DETERMINE SIZE AND SPACING OF GROUT
PORTS BASED ON FLOWABLE FILL PROPERTIES
SHIMS MAY BE USED IN LIEU OF LEVELING BOLTS (ALLOW ALTERNATES ON PLANS)



TYPICAL SECTION: APPROACH SLAB END WITHOUT SLEEPER SLAB

- NOTES:
1. EXCAVATE TRENCH UNDER END OF APPROACH SLAB END PRIOR TO INSTALLATION. THE WIDTH OF EXCAVATION TO BE BASED ON ALLOWABLE SOIL BEARING VALUES.
 2. SET APPROACH SLAB ON GRADE. ADJUST ELEVATION USING LEVELING DEVICE. SHIMS MAY BE USED IN LIEU OF LEVELING BOLTS (ALLOW ALTERNATES ON PLANS)
 3. FILL VOID UNDER SLAB END WITH CONTROLLED DENSITY FILL THROUGH EXCAVATION AND PORTS IN SLAB.
 4. CONSTRUCT APPROACH ROADWAY AND OVERLAY (IF REQUIRED)

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SUGGESTED GUIDE DETAILS PRECAST APPROACH SLABS

SURFACE APPROACH SLAB DETAILS

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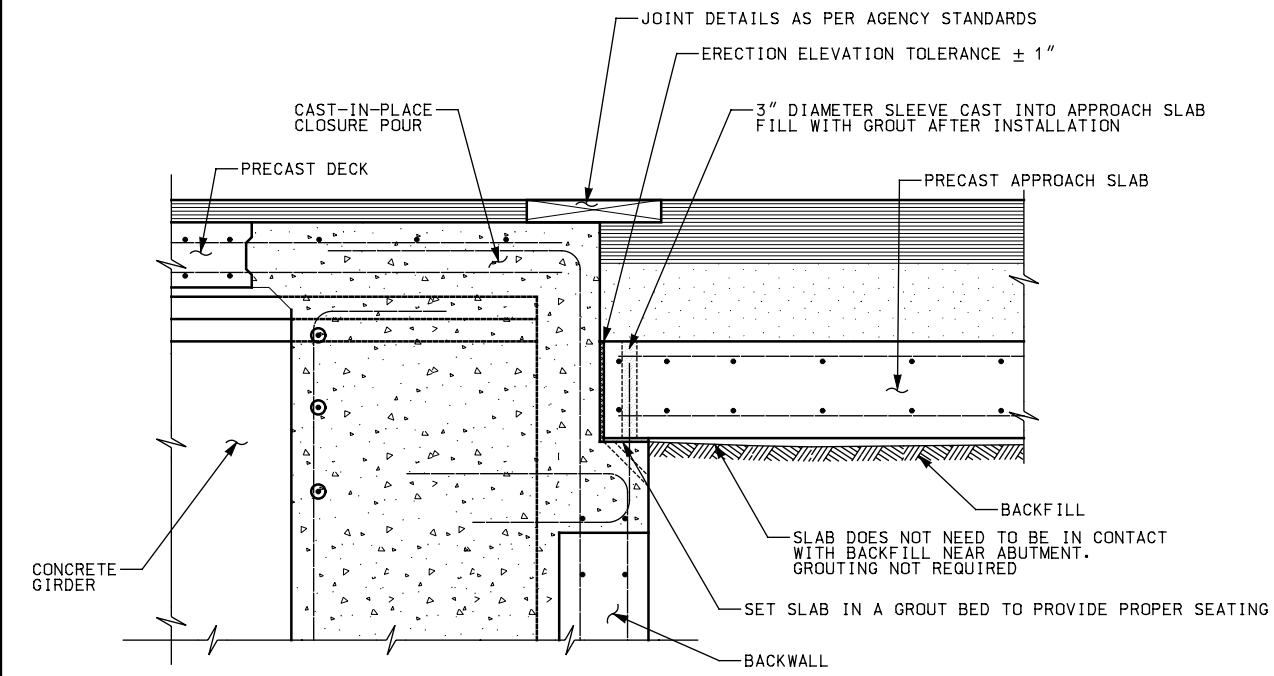
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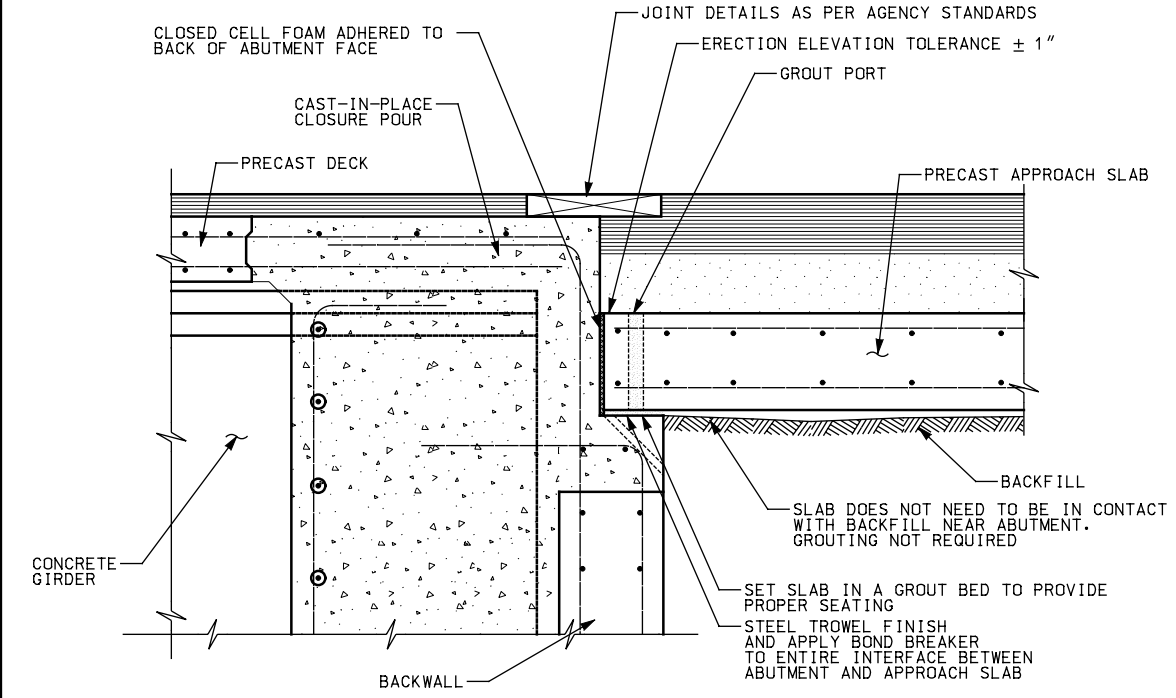
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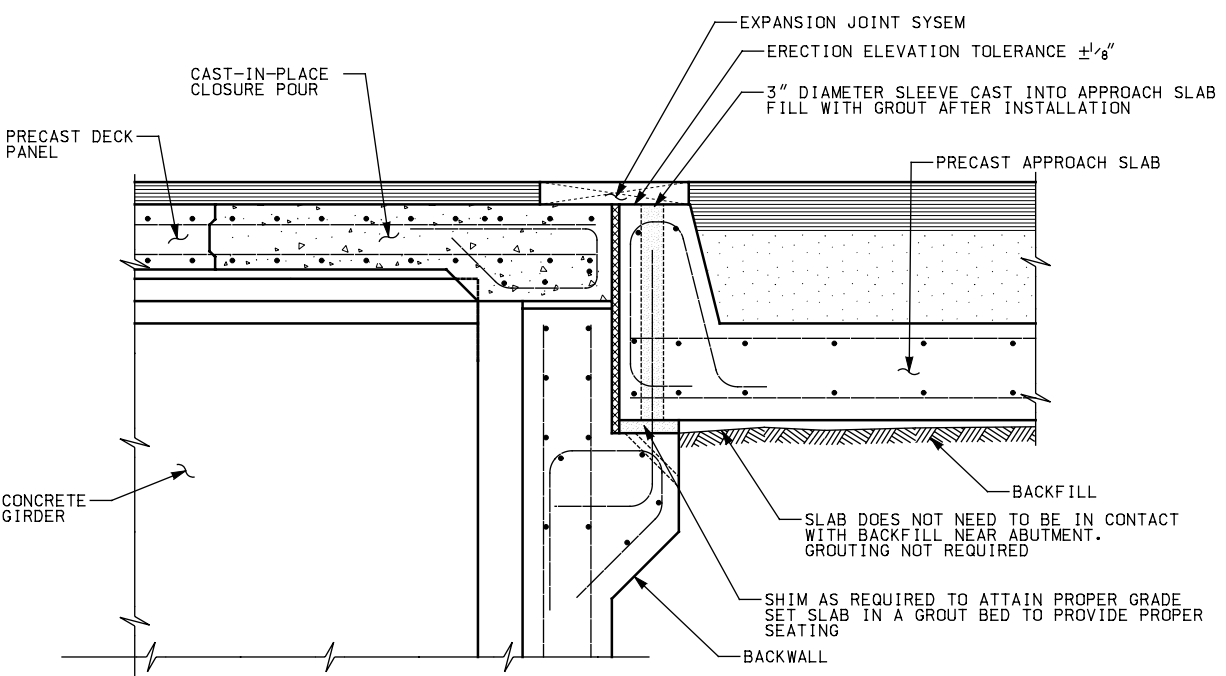
TYPICAL SECTION: SUB-SURFACE APPROACH SLAB AT INTEGRAL ABUTMENT - PINNED CONNECTION

NOTES: CONCRETE GIRDER SHOWN, STEEL BEAM SIMILAR.
NOT ALL INTEGRAL ABUTMENT DIAPHRAGM REINFORCING SHOWN



TYPICAL SECTION: SUB-SURFACE APPROACH SLAB AT INTEGRAL ABUTMENT - SLIDING CONNECTION

NOTES: CONCRETE GIRDER SHOWN, STEEL BEAM SIMILAR.
NOT ALL INTEGRAL ABUTMENT DIAPHRAGM REINFORCING SHOWN



TYPICAL SECTION: SUB-SURFACE APPROACH SLAB AT CANTILEVER ABUTMENT

NOTE: CONCRETE GIRDER SHOWN
STEEL BEAM SIMILAR.

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A	LENGTH	$\pm 1/4''$
B	WIDTH (OVERALL)	$\pm 1/4''$
C	DEPTH (OVERALL)	$\pm 1/4''$
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	$\pm 1/2''$
E	LOCATION OF LEVELING BOLTS	$\pm 1''$
F	SWEEP OVER MEMBER LENGTH:	$\pm 3/8''$
G	LOCATION OF PROJECTING REINFORCING MEASURED FROM A COMMON REFERENCE POINT	$\pm 1/2''$
H	LOCAL SMOOTHNESS OF ANY SURFACE	$\pm 1/8''$ IN 10 FEET
I	ERECTION ELEVATION TOLERANCE (SURFACE APPROACH SLABS ONLY)	$\pm 1/8''$
K	LOCATION OF BLOCKOUTS	$\pm 1/2''$



A	LENGTH	$\pm 1/2''$
B	WIDTH (OVERALL)	$\pm 1/4''$
C	DEPTH (OVERALL)	$\pm 1/4''$
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	$\pm 1/8''$ PER 12 INCH WIDTH $\pm 1/2''$ MAXIMUM
E	VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	$\pm 1/8''$ PER 12 INCH WIDTH $\pm 1/2''$ MAXIMUM
F	SWEEP OVER MEMBER LENGTH:	$\pm 1/2''$
G	LOCATION OF LEVELING DEVICE OR GROUT PORT MEASURED FROM A COMMON REFERENCE POINT	$\pm 2''$
H	LOCAL SMOOTHNESS OF ANY SURFACE	$\pm 1/8''$ IN 10 FEET
I	ERECTION ELEVATION TOLERANCE	$\pm 1/8''$



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