Development of Earthquake-Resistant Precast Pier Systems for Accelerated Bridge Construction in Nevada

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Task 2 – Evaluation Methodology and Rating of Connection Types

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Task 2. Evaluation Methodology and Rating of Connection Types

2.1. Introduction

Task 2 was aimed at (a) developing a methodology to evaluate different connection types that are potentially appropriate for use in accelerated bridge construction (ABC) and suit the needs of NDOT, and (b) rating each connection type using this methodology. Five connection types were evaluated including: grouted ducts, mechanical bar couplers, pipe pin connections, pocket connections, and rebar hinge connections. The evaluation method took into account the level of development [referred to as Technology Readiness Level (TRL)], seismic performance, and non-seismic parameters associated to each connection type. The non-seismic parameters were comprised of potential time saving, initial cost, constructability, inspectability, reparability, and durability. Different rating scales were assigned to each evaluation parameter to simplify the comparison of the connections with respect to that parameter.

This task began by defining and scaling the evaluation parameters, followed by evaluating different connection types based on the proposed rating criteria, and concludes with quantitative comparison of the connections. Sketches of different alternatives for precast multi-column bents with non-integral drop caps, column-inverted-T cap precast connections, and column-integral drop cap precast connections are shown in Appendix A.

2.2. Technology Readiness Level

TRL assesses connections based on their readiness for implementation in ABC in seismic areas and identifies gaps in the knowledge and development of a connection type. Nine levels of readiness developed by Marsh et al. (2011) were utilized to describe the important characteristics of seismic ABC connections. The cumulative TRL rating is defined as the number of satisfied TRL ratings for each connection type, which implies that equal weight is assigned to each TRL parameter.

Table 2-1 summarizes the definition and description of each TRL.

- TRL-1 indicates that the connection concept exists.
- TRL-2 shows that the connection type has been either analyzed or tested for the static strength.
- TRL-3 indicates that the connection is constructible and has been deployed in a low-seismic region.
- TRL-4 indicates that the connection type has been analyzed for seismic loading using analytical models. The analysis should include the response to post-yield cyclic loading to evaluate ductility demand and capacity.
- TRL-5 indicates that all applicable or necessary testing of critical components has been completed. It should be noted that TRL-5 does not apply to all ABC connection categories. For example, in connections fabricated using mechanical bar couplers, the performance of the connections relies mostly on the individual coupler, thus TRL-5 is applicable if couplers have been tested. However, the component test in pocket connections is not relevant. A TRL-5 of 1 (full credit) is assigned to the connections in which component testing is not applicable or not necessary because its absence is not a shortcoming.
- TRL-6 refers to testing of a complete connection under inelastic cyclic loading (e.g. a column-footing pipe pin connection).
• TRL-7 indicates that seismic design guidelines for a connection type have been developed based on comprehensive experimental and parametric analytical studies.
• TRL-8 applies when the connection has been constructed in a high-seismic region using seismic detailing.
• TRL-9 indicates that the connection type has performed adequately during a design-level seismic event in the field. This implies that the connection satisfies the seismic performance and constructability requirements when utilized in high-seismic areas. However, because ABC connections in high seismic zones are relatively new with few field employments, none of the evaluated connections can be expected to achieve TRL-9.

2.3. Seismic Performance Evaluation

The seismic performance rating evaluates how an ABC connection performs compared to a CIP connection for a specific seismic zone (low, moderate, and high) (Marsh et al. 2011). The seismic performance scale is presented in Table 2-2. The rating represents the combined effect of the following issues:
• Has the connection been tested experimentally? If so, is the cyclic behavior comparable to CIP behavior?
• Does the connection emulate the seismic performance of a comparable CIP connection with proper seismic detailing?
• Can the strength of column longitudinal bars be developed under cyclic loading?
• Is the connection damage similar to the damage expected from CIP connections (e.g. concrete spalling, bar rupture, bar buckling, etc.)?
• Does the connection allow for strain penetration under inelastic loading?
• Are bar splices limited to regions outside the plastic hinge zone?
• Can axial and shear load capacity be maintained up to drift ratios at least as high as those of the CIP columns?
• Is the connection self-centering after cyclic loading?
• Can damage be limited to the plastic hinge region?
• Does
• Is the energy dissipation potential of the precast connection comparable to that of the CIP connection?
• Does the connection provide adequate deformability and strength for its intended seismic performance (e.g. capacity-protected connection)?

The higher number of the satisfied issues represents the better expected seismic performance of the connection.

2.4. Non-Seismic Evaluation Parameters

The parameters included in the non-seismic evaluation rating of the connections were defined in terms of the potential time saving, initial cost, constructability, inspectability, reparability, and durability. Each category was evaluated with a scale ranging from -2 to +2 relative to the corresponding value for a comparable CIP connection, which is assumed to be 0. Subsequently, the non-seismic evaluation rating associated to each connection type was determined as the summation of the ratings assigned to individual categories. Connections must be rated at least adequately in all six categories. If a connection receives poor ratings in one or two categories, it should gain higher ratings in other categories to compensate for the low rating.
The scale for categories comprising the non-seismic evaluation rating is presented in Table 2-3. The non-seismic evaluation categories are further elaborated in the following sections.

2.4.1. Time Saving

Potential time saving rating measures the possible construction time advantage for a connection type relative to CIP construction. The actual time saving may be influenced by the bridge in which the connection is incorporated and the prevailing construction culture in the state, thus the evaluation is subjective and approximate. The time saving was evaluated with a scale ranging from -2 to +2, as indicated in Table 2-3.

The evaluation was established based on the time estimates needed for each step of the construction of a CIP bent, and a similar bent constructed using ABC connections. The estimated time saving was then calculated as the difference between the time estimate for the CIP and ABC bridge. The connections were evaluated as a whole and not by individual steps in the construction process. A detailed description of this method is available in Appendix H of Marsh et al. (2011).

2.4.2. Initial Cost

The initial cost rating evaluates how the associated costs for fabrication of an ABC seismic connection is compared with that of a corresponding CIP connection. A scale ranging from -2 to +2 was used to evaluate the initial cost (Table 2-3). The following factors are considered in the rating:

- The total cost of the required parts and materials.
- The cost due to requirements for special handling, lifting, or shoring equipment for field installation (e.g. large-capacity cranes).
- The cost due to requirements for detailing and designing special construction tools or parts to facilitate and accelerate the construction procedure (e.g. matching template for aligning the protruded bars of precast columns with grouted ducts).

2.4.3. Construction Risk

The construction risk rating evaluates the difficulty to fabricate, handle, lift, and install a connection type, how the construction procedure may affect the associated quality, cost, and schedule risk relative to a comparable CIP connection. In other words, the construction risk rating assesses the possibility that something might go wrong during construction and detract from quality or impact the schedule. The construction risk rating criteria are presented in Table 2-4. The following issues are considered in the rating (Marsh et al. 2011):

- Complexity of detailing and number of parts.
- Required construction tolerances during component fabrication and field installation.
- Difficulty of labor access, work environment, and work condition.
- Complexity of installation procedure and number of steps.
- Vulnerability to construction mishaps, such as component damage during handling and noncompliant construction procedures, and the availability of inspection and mitigation methods.
- Sensitivity of installation schedule to individual operations, such as grouting and the time for grout to cure.
• Repetitiveness of work and learning curve.

2.4.4. Inspectability

The inspectability rating evaluates the difficulty in post-earthquake inspection of a seismic ABC connection components compared to the same connection constructed using CIP methods. The rating considers the ability to recognize damage by visual inspection and whether methods are available for damage assessment of the critical structural components. The following issues are considered in the inspectability rating (Marsh et al. 2011):

- Can an inspector conclude that no internal damage is present in critical components if no damage is observed by visual inspection?
- Can visual inspection identify a failure of a critical structural component that needs immediate repair or replacement?
- Can damage be assessed with nondestructive evaluation tools?
- Can damage be assessed with minimal need of destruction?

2.4.5. Reparability

The reparability rating assesses the difficulty in post-earthquake repair of a damaged seismic ABC connection relative to that of a corresponding CIP connection considering the following issues:

- Equipment, parts, and materials required for repair process.
- Need for destruction to access the damaged parts.
- The total required time for repair process is compared with that of a CIP connection.

2.4.6. Durability

The durability rating evaluates the durability of an ABC connection compared to that of a connection built using CIP methods under similar typical environmental exposure. The following issues are considered in the durability rating (Marsh et al. 2011):

- Does the connection provide adequate protection of its structural components?
- Does the connection avoid ingress paths for contaminants or water to structural components?
- Is the durability of the connection affected by the quality of construction?
- Can deterioration be detected during routine bridge inspections?

2.5. Evaluation of Connection Types

In this section, different connection types including mechanical bar couplers, grouted ducts, pocket connections, pipe pin connections, and rebar-hinge connections are evaluated in terms of TRL, seismic performance, and non-seismic evaluation parameters, and a rating is assigned to each category based on the rating system defined in Sections 2.2 through 2.4. The applicability of each TRL level to each connection type is indicated by a check mark in TRL evaluation tables.
2.5.1. Grouted Ducts

2.5.1.1. Technology Readiness Level

Table 2-5 lists the TRL evolution for grouted-duct connections. Seismic and non-seismic performance and behavior of grouted duct connections under monotonic and reversal cyclic loading have been investigated in several studies [Matsumoto et al. 2001; Brenes et al. 2006; Steuck et al. 2008; Matsumoto 2009a; Tazarv and Saiidi 2014] and design equations for development length of reinforcing bars in grouted ducts (Matsumoto et al. 2001; Renes (2006); Steuck et al. 2008; Restrepo et al. 2011) and ducts filled with ultra high performance concrete (UHPC) (Tazarv and Saiidi 2014) have been developed. Not all the design equations that have been developed to date for grouted duct connections account for effects of different parameters on the development length of the bar (e.g. duct diameter, duct spacing). A few bridges have been constructed incorporating this connection type in low and moderate seismic regions.

In summary, the grouted ducts receive a TRL of 8.

2.5.1.2. Seismic Performance Potential

Grouted duct connections used to joint precast columns to footings or columns to precast cap beams need to be designed as capacity-protected elements. This connection is expected to remain damage free and essentially elastic, and force plastic deformations into the column. Confinement provided by the duct around the grout and concrete mass around the duct increases the bond strength, hence reducing the development length of the reinforcing bar within the duct and concentrating the bar strain. This effect is more critical to small size bars that have a shorter development length. Local debonding of the bars distributes the strains over the debonded length, thus reducing strain concentration. However, past studies conducted by Pang et al. (2008) and Matsumoto (2009a) showed that the strain concentration of reinforcing bars embedded in regular grouts minimally affects the seismic performance of columns. Use of high-strength grouts or UHPC to fill the ducts can substantially reduce the development length of the bar. This may concentrate the strains at a very short length and result in a premature fracture of the bar under cyclic loading. Column tests by Tazarv and Saiidi (2010) showed that partial debonding of the column bars embedded in footing UHPC-filled ducts can eliminate strain concentration effects.

Some bars are likely not to be centered in their duct during construction. Test results [Raynor et al. (2002); Brenes et al. (2006)] showed that bond strength of off-centered bars slightly differs from that of centered bars, but this may be inevitable in the field.

In summary, past studies showed that grouted-duct connections can emulate the seismic performance of comparable CIP connections and be a viable alternative for ABC constructions. Therefore, this connection type receives a seismic performance rating of 0.

2.5.1.3. Non-Seismic Evaluation Parameters

Adding up the ratings for six non-seismic parameters, the grouted duct connections receive a composite non-seismic rating of 0. The rating evaluation for each individual non-seismic parameter is discussed in the following subsections.

2.5.1.3.1. Time Saving

The time saving potential of grouted ducts compared to CIP construction is evaluated to
be very high, especially when they are incorporated in precast cap beams, thus a rating of +2 is assigned. The total saved time may vary depending on the number of the bent spans. In single column bents, the required time to align the column longitudinal bars with the ducts embedded in a precast cap beam is less than that for multi-span bents. However, the total installation time for multi-span bents will be still shorter than the construction time for CIP cap beams.

2.5.1.3.2. Initial Cost

Use of grouted-duct connections requires precast fabrication of one or both of the adjoining members. Therefore, large capacity cranes may be required depending on the precast member type. If grouted ducts are embedded in the cap beam, construction costs related to shoring and securing the cap beam will be eliminated because the cap beam can be placed on top face of the columns prior to grouting. The grout type (or UHPC) and grouting procedure may impose substantial costs compared to CIP construction, although the required volume of the grout material may be small. For example, use of UHPC may cost 22 times more than that of regular grouts, which can be significant even when a small volume is used. Because visual inspection of the duct or interface to be grouted is generally not possible, methods for ensuring complete filling are necessary.

Overall, initial costs associated with implementation of grouted-duct connections may be higher than CIP construction, hence a rating of -1 is assigned.

2.5.1.3.3. Construction Risk

Grouted duct connections are associated with moderate construction risk in most circumstances and are assigned a risk rating of -1, implying a slightly higher risk than CIP construction. The major variables to consider are: (1) the number of bars, (2) the size of the ducts relative to the bars (tolerance), (3) the number of precast elements to be joined simultaneously, and (4) the location of the connectors within the cross section.

The protruding bars and embedded ducts should be located correctly for aligning the protruded bars with the ducts in the adjoining member (footing or cap beam). This alignment is facilitated by using the same template for placing the bars in one element and ducts in the other element. When connection bars are located near the column center (e.g. to form a hinge connection), accurate placement of the central bars is more difficult because the bars cannot be secured to the column bar cage, and therefore, special measures may be required. When the ducts are incorporated in columns, a thicker than normal longitudinal bar concrete cover is needed to satisfy the required cover around the ducts. Furthermore, different confining spirals than those of a column bar cage need to be provided around the ducts. Use of fewer but larger bars may speed up and simplify the installation process; however, the maximum bar size may be limited by the required embedment length and spacing between the reinforcing bars. Fixed location of the ducts embedded in precast cap beams of multi-span bents makes meeting the tolerances more difficult in comparison to single column bents because alignment of the bars and ducts should be provided at multiple locations at the same time.

Placement of the grouting ducts in footing or precast cap beams requires that either pressure grouting be performed from the bottom of the ducts using embedded grouting tubes or the ducts be filled with grout prior to placement of the precast column. The latter procedure is preferred for UHPC due to high viscosity of the material. Grouting the ducts from the bottom requires additional work and introduces the risk of the tubes being damaged while casting the footing or cap beam. Complete filling of the ducts with grout (or UHPC) is crucial to
performance of this connection type; therefore, pressure grouting is preferred. The footing ducts may be filled with water or debris and need to be cleaned out prior to grouting. This issue is particularly critical when the ducts are grouted from the top because if the ducts clog or become filled with debris, the column longitudinal bars cannot fully fit into the grouted ducts and hence the operation is interrupted. Preferably, grouting ducts are not placed in the columns because it inevitably places the spliced bars in the plastic hinge region of columns, which may affect the seismic performance.

2.5.1.3.4. **Inspectability**

The inspectability of grouted-duct connection after a damaging seismic event is similar to that of CIP connections, therefore a rating of 0 is assigned. To detect fractured bars in both connection types, some concrete would have to be removed.

2.5.1.3.5. **Reparability**

The durability of grouted-duct connections is about the same as that of CIP connection. Therefore, a rating of 0 is assigned. The susceptibility to water penetration and consequent corrosion of these connection types is generally similar. However, unlike the cold joints forming in CIP concrete pours, a grout pad is placed at the interface of precast and adjoining members. If the grouting process does not fully eliminate the likelihood of formation of air voids in the grout, the durability of the grouted ducts may be inferior to that of CIP connections.

2.5.1.3.6. **Durability**

The durability of grouted-duct connections is about the same as that of CIP connection. Therefore, a rating of 0 is assigned. Unlike the cold joints forming in CIP concrete pours, a grout pad is placed at the interface of precast and adjoining members. The susceptibility to water penetration and consequent corrosion of these connection types is generally similar. If the grouting process does not fully eliminate the likelihood of formation of air voids in the grout, the durability of the grouted ducts may be inferior to that of CIP connections.

2.5.2. **Mechanical Bar Couplers**

The evaluation is performed for five types of connections fabricated using mechanical bar couplers including bar-grip, grouted-sleeve, headed-bar, shear-screw, and threaded couplers.

2.5.2.1. **Technology Readiness Level**

Based on the literature search performed in Chapter 1, the TRL evaluation of the mechanical bar couplers is summarized in Table 2-6. This table shows the level of development that has been accomplished for each type of coupler. The evaluation relates to application of connectors in plastic hinge zone of bridge columns. Except for grouted-sleeve and headed-bar couplers, none of the coupler types have been incorporated in plastic hinge zones of actual bridge columns in moderate and high seismic areas. The headed-bar couplers were used to splice SMA fuse bars in plastic hinge regions of WashDOT SR-99 North-Bound Off-Ramp bridge columns in Seattle, Washington. Grouted sleeves are more commonly used than other types of couplers, and have been utilized in several projects in Utah. Additionally, grouted sleeves are part of the Utah DOT Precast Substructure Elements Manual (2010).
In summary, the bar-grip, grouted-sleeve, headed-bar, shear-screw, and threaded couplers receive TRL ratings of 4, 8, 6, 4, and 5, respectively.

2.5.2.2. Seismic Performance Potential

Results of in-air component tests and subassembly tests incorporating mechanical couplers have shown promising results in terms of displacement ductility, energy dissipation, and ultimate capacity. Most of the test results are related to in-air component tests of couplers. Only a relatively small number of tests on column connections using couplers have been conducted. However, in-air test results are available for all coupler types.

Due to limited available test data on seismic performance of bar-grip couplers used in column plastic hinges, evaluation of this type of couplers is not possible at time of writing this document.

In-air tensile tests of grouted-sleeve couplers have shown that the dominant failure mode for these couplers might be bar fracture away from the coupler region, bar pullout from the coupler, or coupler fracture. When one end of the coupler is threaded (referred to as threaded-grouted-sleeve coupler), tests have shown that the thread region is the common failure location. The average yield and ultimate load capacity of grouted-sleeve couplers are comparable to those measured for reference bars, and grouted-sleeve and threaded-grouted-sleeve couplers can satisfy AASHTO LRFD (2014) fatigue requirements. Columns constructed using grouted-sleeve splices in plastic hinge regions and loaded cyclically exhibited lower displacement ductilities compared to CIP columns. Although the minimum displacement ductility requirement of 5 specified by the codes (AASHTO LRFD 2014; Caltrans SDC 2010) can be achieved depending on the connection detailing, the reserve displacement capacity is relatively small. Performance of grouted-sleeve connections can be improved by debonding longitudinal bars outside the coupler region to minimize the strain concentration outside the coupler. Due to uncertainties in the failure mode and weaker seismic performance of grouted-sleeve connections compared to CIP connections, a rating of -1 is assigned.

Results of the in-air monotonic, cyclic, and dynamic tests of headed-bar couplers have shown that spliced bars could achieve the ultimate stress and strain capacities while the coupler region undergoes large plastic deformations. The large strain capacity of these couplers can be beneficial when they are employed in plastic hinge regions. Cyclic tests of columns utilizing headed-bar couplers to link the reinforcing bars of the column and footing at plastic hinge areas have demonstrated that the columns perform seismically similar or better than comparable CIP columns and have large displacement ductility capacity suited for high seismic regions. Thus, a rating of 0 is assigned to headed-bar couplers.

Columns with long shear-screw couplers exhibit seismic performance comparable to that of CIP columns. Tensile strength of shear-screw couplers may be sufficient for seismic applications depending on the product and manufacturer. However, these couplers generally limit the strain capacity of the bars due to strain concentration beneath the screws. This limitation is critical for seismic applications, especially in the plastic hinge area where large plastic deformations are expected. Therefore, a rating of -1 is assigned to shear-screw couplers.

Tensile tests of bars spliced using taper threaded couplers have shown inferior performance, premature failure, and low strain capacity of this coupler type. However, the ultimate strength and strain capacities of the bars were developed when straight threaded couplers were utilized. The columns that incorporated straight threaded couplers in plastic hinge regions to link the column and footing bars showed equal or improved drift ratio capacity
compared to reference columns, confirming suitability of application of straight threaded couplers in the plastic hinge of columns located in high seismic regions. Therefore, a rating of 0 is assigned to straight threaded couplers.

2.5.2.3. Non-Seismic Evaluation Parameters

Adding up the ratings for the six comprising components of the non-seismic evaluation parameter, the bar-grip, grouted-sleeve, headed-bar, shear-screw, and straight threaded couplers receive composite non-seismic ratings of -1, 0, 0, -1, and -1, respectively. The rating evaluation for each individual non-seismic parameter is discussed in the following subsections.

2.5.2.3.1. Time Saving

The time saving for bar coupler connections is rated as +2 because incorporation of mechanical bar couplers allows for precast fabrication of bridge elements. The majority of time saving comes from construction of cap beam as a precast element.

2.5.2.3.2. Initial Cost

Incorporation of mechanical bar couplers requires special measures to lift the precast column, secure it in place, and align it with connector bars of the other member when couplers are used at the bases of columns. Of the connection types, the one with shear-screw couplers needs more labor to install the fuse bar in between the adjoining members. Bar-grip couplers require a special installation machine to couple the bars, which may delay the installation process due to difficulty in access to the bars. Depending on the dominant construction culture of a specific area, the installation and handling process and costs may vary among couplers. Nonetheless, installation of couplers, regardless of the type is expected to increase the construction cost compared to CIP construction. Thus, a rating of -1 is assigned to mechanical bar couplers.

2.5.2.3.3. Construction Risk

The variables to be considered in constructability of a connection are: (1) the number of bars, (2) the number of precast elements to be joined, and (3) the location of the connection within the cross section. Projecting bars from a precast element need to be aligned with protruded bars in the other member. The alignment tolerance for connections with threaded parts (headed-bar or threaded-grouted-sleeve couplers) is tight.

Misalignment of interior bars used for bar hinge connections is more critical because it is difficult to secure hinge bars to the column cage during construction. To provide the required cover concrete around the couplers, column longitudinal bars needs to be shifted slightly inward. Furthermore, different confining spirals in coupler zone may be required. These two requirements are more limiting for grouted sleeves compared to the other coupler types owing to the relatively large diameter of the coupler. To reduce the construction work, fewer, larger bars can be used; however, the bar size may be limited by the required development length of the bar.

Grouted sleeves allow for higher tolerances [approximately +/- 0.5 in. (12.7 mm)] relative to other types of mechanical couplers. Oversize sleeves allow for larger tolerances; however, the sleeve size may be limited by the required spacing between the precast member longitudinal bars. Grouted sleeves embedded inside the adjoining members (footing or cap beam) in lieu of precast columns require tighter tolerances due to fixed location of the sleeves.
Use of couplers to connect columns to cap beams in multi-column bents is more difficult than single-column bents in terms of bar alignment and required tolerances. The construction risk of grouted-sleeve couplers is slightly higher than CIP connections but is lower than the other coupler types, because: (1) they allow for higher tolerances, (2) contrary to other types, they have been deployed in several bridge projects in the low-to-moderate seismic areas, and (3) there is a construction manual available for this coupler type (Utah DOT Precast Substructure Elements Manual 2010). Therefore, a construction risk ratings of -1 and -2 are assigned to grouted couplers and other coupler types, respectively.

2.5.2.3.4. Inspectability

The post-earthquake inspectability of bar coupler connections is similar to that of CIP connections; therefore, a rating of 0 is assigned. In both cases, an investigation for fractured bars (or couplers) will probably require removal of some concrete to allow for visual inspection. In that case, the procedure will be less damaging if the bars are located near the surface of the element than in the middle of the cross section.

2.5.2.3.5. Reparability

Repair of grouted-sleeves, threatet grouted-sleeve, shear-screw, and bar-grip couplers is very difficult unless the damaged couplers are removed and replaced with bar segments spliced using bar-grip or shear-screw couplers. The repair process of the headed-bar couplers is relatively easy because the bar could be readily replaced with a new one. However, in all repair strategies, the damaged elements need to be realigned to allow for installation of the new bars. A rating of 0 is assigned to all mechanical couplers except for headed-bar coupler that receives a rating of +1.

2.5.2.3.6. Durability

If the coupler is susceptible to moisture penetration, types that transfer the loads through threads or contact of metallic parts are more likely to suffer from corrosion. However, sufficient cover concrete can prevent moisture penetration to the couplers. At the connection between a column and adjoining member, there may be a grout pad at the precast member interface instead of the cold joints formed in CIP construction. Susceptibility of these two joint types to water ingress is generally similar. Thus, a durability rating of 0 is assigned to all couplers.

2.5.3. Pocket Connections

As discussed in Section 1.2.3.1, pocket connections are constructed either by extending the longitudinal bars of a precast column into a pocket and filling the pocket with concrete or grout (referred to as “ELB-PC” hereafter) or extending a precast column into the pocket and grouting the gap between the column and the pocket (referred to as “EPC-PC” hereafter). In this section, both connection types at the bottom (denoted by “B”) or top (denoted by “T”) of columns are evaluated separately.

2.5.3.1. Technology Readiness Level

Based on the literature search carried out in Chapter 1, the TRL evaluation of pocket connections is presented in Table 2-7.
The ELB-PC has been used in several field practices as shown in Figs. 1-88 through Fig. 1-94. However, most of the prior implementations have been limited to low-seismic areas. Seismic performance of the column-cap beam ELB-PC has been investigated in studies by Matsomoto (2009b and 2009c), Restrepo et al. (2011), and Mehraein and Saiidi (2015). Based on the research by Matsomoto (2009b and 2009c) and Restrepo et al. (2011), a design guideline for use of this connection type in precast cap beams has been developed. The seismic performance of the base ELB-PC has not been studied either experimentally or analytically at the time of writing of this report. Anchorage strength of bars grouted into pockets in a beam has been studied in monotonic tests conducted by Matsumoto et al. (2001).

The EPC-PC has been employed in low- to high-seismic areas including a two-span bridge in Washington as a part of WSDOT highways for life (HFL) project, in which precast columns were connected to footings using pocket connections. Examples of the field deployment of EPC pocket connections can be found in Section 1.2.3.1. Seismic performance of EPC-PC has been studied by Zhu et al. (2006), Haralsson et al. (2011), Motaref et al. (2011), Ziehl et al. (2011), Tran and Stanton (2012), Kavianipour and Saiidi (2013), Larosche et al. (2013), and Mehrsoroush and Saiidi (2014). Based on the studies, design recommendations have been proposed for detailing and embedment length of the column in the pocket enabling them to satisfy the code requirements for capacity protected moment connections. The absence of testing of critical components in TRL evaluation of EPC-PC is not regarded as a negative point because essentially no unusual components are incorporated in this connection type.

In summary, ELB-PC-B, ELB-PC-T, EPC-PC-B, EPC-PC receive TRL ratings of 2, 6, 8, and 8, respectively.

2.5.3.2. Seismic Performance Potential

The pocket connection is intended to behave as capacity a protected components meaning that all the nonlinearities has to take place in the column while the connection itself remains essentially elastic and damage free. Thereby, the seismic response of the connection is expected to emulate that of a comparable CIP connection.

Seismic performance of ELB-PC-T has been investigated in connection subassembly tests by Matsomoto (2009b and 2009c) and Restrepo et al. (2011), and two, two-column bent tests by Mehraein and Saiidi (2015). In the connection subassembly tests, the column longitudinal bars passed through the pockets that extended within the entire height of the member. However, in the bent tests, cap beam longitudinal bars were bundled to the sides and the pocket was formed in the cap beam using steel metal pipes. Cyclic tests demonstrated that the connection performance emulated CIP moment connection when column longitudinal bars are developed adequately in the pocket and the joint region are detailed sufficiently to prevent joint shear failure. Therefore, ELB-PC-T receives a seismic performance rating of 0.

Seismic performance of column-footing EPC-PC-B has been studied in single-column tests by Zhu et al. (2006) and Haralsson et al. (2011), and in two-column pier and a four-span bridge tests by Motaref et al. (2011) and Kavianipour and Saiidi (2013), respectively. In addition, Mehrsoroush and Saiidi (2014), Ziehl et al. (2011) and Larosche et al. (2013 and 2014), and Tran and Stanton (2012) have investigated the seismic response of column-cap beam, pile-pile cap, and column-pile shaft EPC-PC-T, respectively, under cyclic reversal loading. Test results demonstrated that the seismic performance of EPC-PC (B and T) is as good as that of a comparable CIP connection when embedment length of the column in the pocket and the joint detailing are sufficient to form the plastic hinge in the column. Therefore, a seismic performance
rating of 0 is assigned to EPC-PC (B and T).

2.5.3.3. Non-Seismic Evaluation Parameters

Adding up the ratings for the six comprising components of the non-seismic evaluation parameter, ELB-PC-B, ELB-PC-T, EPC-PC-B, EPC-PC receive composite non-seismic ratings of -2, +1, +1, and +3, respectively. The rating evaluation for each individual non-seismic parameter is discussed in the following subsections.

2.5.3.3.1. Time Saving

ELB-PC and EPC-PC incorporated at column tops receive a time saving rating of +1 and +2, respectively. The associated ratings for pocket details at the bottom of column are 0 and +1. The majority of time saving is due to precasting the elements, especially cap beams. Thus, the total time saving associated to column-cap beam pocket connections is more than the time saving for column-footing pocket details. The pocket in ELB-PC may be filled either by grout or concrete. Use of concrete reduces the construction speed due to the longer required curing time compared to grout. Columns are required to remain secured until the infill material has gained sufficient strength in the pocket. The time saving potential is reduced for ELB-PC relative to EPC-PC connections, because the pocket diameter in ELB-PC cap beams is larger than that of columns, which necessitate shoring the cap beam. For EPC-PC, the cap beam may be placed on the top face of the columns with some spacers in between, eliminating the need for shoring.

2.5.3.3.2. Initial Cost

Initial cost ratings of -1 and +1 are assigned to ELB-PC at the top and bottom, respectively. The use of pocket connections relies mostly on the precast fabrication of structural elements. Hence, large capacity cranes may be required depending on the weight and size of precast members. Implementation of column-footing pocket connections using ELB-PC requires special measures to hold the precast column in place over the infilled pocket in the footing. As discussed in Section 2.5.3.3.1, the cap beam is required to be shored and secured temporarily when column is connected to cap beam, adding to the cost of ELB-PC-T. Nonetheless, due to the higher labor and formwork cost, the construction cost of CIP cap beams is higher than that of ELB-PC-T.

The EPC-PC-B and EPC-PC-T receive ratings of 0 and +1. For EPC-PC, costs associated with required parts, materials, and equipment are comparable to those of CIP moment connections. At the bottom, the extended precast column is readily placed in the pocket and the gap is grouted. However, at the top, the cap beam can either be shored or held on top of the columns depending on the presence of the slab on top of the pockets in the cap beam. The associated construction cost of both precast cap beams is lower than a comparable CIP cap beam because precast fabrication requires lower labor and formwork cost. Sealing of the gap that forms between the column and cap beam prior to casting the pocket also introduces some extra costs; however, these costs are much lower than the ones for construction of the cap beam itself. However, the associated costs of the sealing is not comparable to those of the cap beam itself.

2.5.3.3.3. Construction Risk

ELB-PC offers the same construction risk as that of CIP moment connections when reinforcing bars are bundled to the sides of the pocket, hence a construction risk rating of 0 is
assigned. However, when reinforcing bars of the footing or cap beam pass through the pocket, the construction risk will be higher as discussed in the following. On the contrary, the construction risk for EPC-PC is lower than comparable CIP connections, thus EPC-PC receives a rating of -1.

Construction of EPC-PC requires fewer steps compared to ELB-PC. EPC-PC-B needs the precast column extending into the pocket to be secured, while EPC-PC-T needs the cap beam to be either placed on top of the column or shored depending on the depth of the pocket. However, for ELB-PC-B, the precast column with extended longitudinal bars into the precast footing pocket should be held over the pocket up to curing the pocket infilled material. For ELB-PC-T, the cap beam needs to be shored temporarily until the pocket infilled material has gained sufficient strength. The gap between the cap beam and column should be sealed tightly to prevent seepage of the grout or concrete.

Precast column fabrication, handling, and transportation are simplified for EPC-PC relative to ELB-PC because there are no protruding reinforcing bars. The development length of the column longitudinal bars in pockets needs to be provided through embedment length of straight bars or headed anchors when the pocket height is short.

Construction tolerances are similar for both types of pocket connections. The precast column needs to be aligned with the pocket prior to filling the pocket with concrete or grout in ELB-PC or grouting the gap in EPC-PC. Both connections provide large tolerances during site erection when connecting either columns to footings or columns to precast cap beams in single-column bents. The precast column can be shifted laterally inside the pocket to correct the column alignment. However, the fixed location of columns and pockets in precast cap beams of multi-span bents requires relatively tight tolerances because the columns and pockets should be aligned at multiple locations. When cap beam or footing longitudinal bars pass through the pocket, the construction tolerances are also affected by the size and spacing of the cap beam and column bars. The vertical bars from the column must be aligned to avoid any cap beam reinforcement crossing through the pocket.

The pocket in footing ELB-PC may be cast with concrete or grout either prior to holding the precast column over the pocket and extending the protruded bars into the precast pocket or after extending the column longitudinal bars into the pocket. However, the pocket in cap beam of ELB-PC-T is cast similarly to the latter. In EPC-PC, the gap between the column and the pocket is grouted either by filling the gap with grout or pressure grouting from the bottom of the pocket using embedded grouting tubes. Complete filling of the gap with grout is crucial to the performance of this connection type. Grouting the pockets from the bottom requires additional works and introduces the risk that the tubes will be damaged while casting the footing.

2.5.3.3.4. Inspectability

The post-earthquake inspectability of pocket connections is the same as that of CIP connections, hence an inspectability rating of 0 is assigned.

2.5.3.3.5. Reparability

ELB-PC and EPC-PC receive a reparability rating of 0. As mentioned in Section 2.5.3.2, pocket connections are detailed to force the inelastic actions to occur in the column and not in the connection zone. However, if any damage occurs in the pocket or embedded portion of column in the pocket, the accessibility to the damaged area will be similar to that of CIP connections.
2.5.3.3.6. Durability

ELB-PC and EPC-PC receive durability ratings of -1 and +1, respectively. A grout pad may be placed at the interface of the column and the adjoining member in ELB-PC (B and T). Local voids in the grout pad may cause the durability of ELB-PC to be inferior to that of CIP connections. On the contrary, in EPC-PC, the column extends into the pocket with no need for a grout pad at the interface. In addition, there is no reinforcing steel crossing the cold joint at the interface, which is an improvement over CIP.

If the top of the pocket detail is protected by a CIP diaphragm, the durability will be comparable to that of CIP connections. Otherwise, the cold joint forming around the perimeter of the pocket increases the likelihood of water or corrosive agents ingress.

The grouting process needs to fully eliminate the likelihood of formation of air voids in the grout, otherwise the durability of pocket connections may be worse than that of CIP connections.

2.5.4. Pipe Pin Connections

In this section, the evaluation parameters for the top and base pipe pin connections are discussed and rated.

2.5.4.1. Technology Readiness Level

TRL evaluation of top and base pipe pin connections is presented in Table 2-8. Tests on individual components of pipe pin connections have been conducted by Doyle and Saiidi (2008) and Zaghi and Saiidi (2010) to investigate the general response of pins under pure shear and combined shear and flexure, bearing strength of pipes against concrete, and pure shear strength of the pipes.

The seismic performance of top and base pipe pins has been explored through shake table tests and cyclic loading tests of large-scale bents incorporating each type of pin (Zaghi and Saiidi 2011; Mehrsoroush and Saiidi 2014). In addition, the ultimate capacity and dominant modes of failure of base pipe pins under direct tension have been investigated by Saiidi and Mehrsoroush (2014). Analytical studies of the test models have also been performed to further investigate the proof of concept and determine the validity of the modeling assumptions based on the correlation between the experimental and analytical results. On the basis of experimental and analytical studies, design guidelines have been developed for both pin types. Due to the satisfactory performance of the top pins, several tests have been carried out at the University of Nevada, Reno (Motaref et al. 2011; Kavianipour and Saiidi 2013; Varela and Saiidi 2015), following the design procedure developed in previous studies.

While base pipe pins have not been used in practice, top pipe pin connections have been employed in several CIP bridges including the approach ramps of the replacement of San Francisco-Oakland Bay Bridge. Although top pipe pins are yet to be utilized in ABC projects, TRL-8 is credited in Table 2-8 because no difference is expected between the CIP and ABC versions of top pipe pins.

In summary, the base and top pipe pins receive TRL ratings of 6 and 7, respectively.

2.5.4.2. Seismic Performance Potential

Because CIP rebar hinge connections are substantially more common than pipe pin connections, seismic performance of base and top pipe pin ABC connections is evaluated in
comparison with that of CIP rebar hinges.

Top pipe pin connections fully eliminate moment transfer between the column and cap beam. However, formation of some moment at base pipe pins is inevitable due to presence of tension members (Mehrsoroush and Saiidi 2014). Both connection types are designed to satisfy the requirements of the codes (AASHTO 2014; Caltrans 2013) for capacity protected elements, meaning that the pins are required to remain damage free and essentially elastic after a seismic event. Contrary to pipe pins, rebar hinge connections undergo large nonlinearities - indicative of damage - while resisting the seismic forces, such as cracking and shear sliding of the connection and yielding of the longitudinal hinge bars. Some level of moment is also transferred by rebar hinges, regardless of whether the connection is at the top or bottom of the column.

Based on the studies conducted by Motaref et al. (2011), Kavianipour and Saiidi (2013), and Varela and Saiidi (2015) seismic performance of top pipe pins constructed based on ABC methods fully emulates that of CIP pipe pins.

Overall, top and base pipe pins receive a rating of +2 and +1, respectively.

2.5.4.3. Non-Seismic Evaluation Parameters

Adding up the ratings for the six comprising components of the non-seismic evaluation parameter, the base and the top pipe pins receive composite non-seismic ratings of +1 and +2, respectively. The rating evaluation for each individual non-seismic parameter is discussed in the following subsections.

2.5.4.3.1. Time Saving

Pipe pin connections can be constructed based on the ABC and CIP techniques. However, deployment in ABC relies mostly on precast fabrication of structural elements. The majority of time saving for top pipe pins is realized when a precast cap beam is used. Incorporation of base pipe pins in ABC requires the column to be constructed as a hollow precast element and then either be filled partially or completely depending on the column type (hollow vs. solid). The total time saving for pipe pins constructed based on ABC methods compared to CIP rebar hinge connections is evaluated to be high. Therefore, a rating of +2 is assigned.

2.5.4.3.2. Initial Cost

The initial cost for pipe connections is rated as 0. As discussed in Section 2.5.4.3.1, use of pipe pins in ABC requires precast fabrication of structural elements. Hence, large capacity cranes may be required depending on the precast member type. Costs associated with required parts, materials, and installation equipment are comparable to those of CIP rebar hinge connections. The installation cost does not vary significantly from CIP rebar hinges.

2.5.4.3.3. Construction Risk

The inner and outer pipes in base pipe pins or the pipe and the can in the top pipe pins should be aligned for proper fitting. To fabricate the base pipe pins based on either ABC or CIP construction methods, the inner pipe extends out of the footing into an outer pipe that is placed over a ring plate on top of the footing. Then, the tension member of the connection is anchored on the top face of the outer pipe using a bearing plate. Because erection of the base pipe pin is done prior to casting of concrete inside the column, the base pipe pins receive a rating of 0.
Fabrication of the top pipe pins based on ABC techniques requires the cans to be placed in a precast cap beam with a relatively tight tolerance. The fixed location of the cans in the cap beam of multi-column bents further requires that tight tolerance limits be met for the pipes and the cans to fit. However, this is not an issue in CIP construction because the can is centered with the pipe prior to casting the cap beam.

Overall, a construction risk rating of -1 is assigned to top ABC pipe pins.

2.5.4.3.4. Inspectability

Inspectability ratings of +1 and 0 are assigned to top and base pipe pins, respectively. As mentioned in Section 2.5.3.2, pipe pins are designed as capacity protected elements meaning that no damage is expected during a seismic event. Nonetheless, the post-earthquake inspectability of top pipe pins may be better than comparable CIP connections because the cap beam can either be jacked up or removed to inspect the can, hinge gap, and the protruded length of the pipe out of the column. However, visual inspection of the pipe in the column requires some removal of concrete. Base pipe pins cannot be dismantled to inspect the pipes and the tension member. In comparison, rebar hinge connections cannot be inspected easily because they are difficult to access. This offers a similar inspectability rating for base pipe pins as that of rebar hinge connections.

2.5.4.3.5. Reparability

Pipe pin connections are expected to remain damage free and essentially elastic during a seismic excitation. Tests by Zaghi and Saiidi (2011b) and Mehrsoroush and Saiidi (2014) showed that the damage to pipe pins was limited to some minor spalling of the column at the column edges caused by hinge gap closure while the pipes remained intact. However, if any unexpected damage is detected, the reparability of top pipe pins will be easier than that of rebar hinge connections for the reason mentioned in Section 2.5.4.3.4. Due to difficulty in access to the damaged parts, the reparability of base pipe pins is comparable to that of rebar hinges. Therefore, reparability ratings of +1 and 0 are assigned to top and base pipe pins, respectively.

2.5.4.3.6. Durability

The durability of pipe pin connections could be a concern unless the steel pipes are properly protected. When the hinge throat consists of concrete, the durability of pipe pins is the same as that of rebar hinges. When the hinge throat is formed by a steel or a rubber ring, sufficient sealing of the connection is required. A rating of -1 is assigned to pipe pins due to concerns about water penetration and subsequent corrosion.

2.5.5. Rebar Hinge Connection

Rebar hinges can be incorporated in ABC utilizing pocket details (Fig. 2-1), mechanical bar couplers (Fig. 2-2), or grouted ducts (Fig. 2-3). The pocket rebar hinges may be formed either in the footing or column. In the former case, the reinforcing bars are embedded at the bottom of a hollow or solid precast column and extended into a concrete or grout filled pocket in the footing (Fig. 2-1a). The latter is constructed by embedding the hinge bars in the footing, extending the protruded portion of the bars out of the footing into a hollow precast column placed over a ring plate, and filling the hollow section with self-consolidating concrete (SCC) (Fig. 2-1b). The filler concrete may be conventional or SCC depending on the height of the
column and the designer’s preference. For erection of the rebar hinges using mechanical bar splices, the couplers can be either embedded in the footing (Fig 2-2a) or the column (Fig. 2-2b) to link the column and footing rebar hinge reinforcements. Grouted ducts may also be utilized to anchor the hinge bars in the footing. The concentrated reinforcing bars of the hinge are embedded at the bottom of a precast column (solid or hollow) and extended into the grout or UPHC filled ducts in the footing (Fig. 2-3).

Rebar hinges constructed using CIP methods are regarded as a reference for ABC pin connections at the top and bottom of columns. Therefore, seismic and non-seismic evaluation parameters for CIP rebar hinges are rated as 0. In this section, the rebar hinges fabricated using ABC methods are evaluated based on comparison with the CIP reference hinges. The seismic performance parameters for the bottom (denoted by “B”) and the top (denoted by “T”) rebar hinges are presented separately. The ratings of the non-seismic evaluation parameters for the ABC rebar hinges are the same as those of the corresponding moment connections for grouted ducts (Section 2.5.1.3), mechanical bar couplers (Section 2.5.2.3), or ELB-PC (Section 2.5.3.3), and are not repeated.

2.5.5.1. Technology Readiness Level

TRL evaluation of CIP and ABC rebar hinge connections is based on the literature search presented in Chapter 1 (Table 2-9).

The CIP rebar hinges have been implemented in several field projects in high seismic areas, including Dunrobin avenue overcrossing in Los Angeles, bent 25C of San Francisco airport viaduct, I-515/I-215 interchange in Las Vegas, and US-395 bridges in Reno. Seismic performance of CIP rebar hinges has been studied by Lim and McLean (1991) and Haroun et al. (1994) under low reverse cyclic loading. Besides, Saiidi et al. (2009) and Mehraein and Saiidi (2015) performed shake table tests and analytical modeling of the top CIP and the base ABC rebar hinges, respectively, to further investigate the seismic response and behavior of this connection type. The absence of critical components testing for rebar hinge connections is not regarded as a negative point because essentially no components are incorporated in this connection type except for reinforcing bars. Caltrans BDS (2003) has special provisions for design of column hinges. Additionally, Cheng et al. (2010) has developed a design guideline for seismic design of rebar hinges.

No information is available on field performance of modern rebar hinges in recent earthquakes. Therefore, TRL-9 is not checked in Table 2-9.

In summary, the CIP and ABC rebar hinge connections receive TRL ratings of 6 and 5, respectively.

2.5.5.2. Seismic Performance Potential

The slow cyclic and shake table tests of the columns connected to footing or cap beam using rebar hinge connections have shown extensive damage to hinge connections as a result of shear sliding at the hinge gap. Under high seismic loads, rebar hinges crack and the longitudinal bars crossing through the gap undergo large plastic deformations resulting in sliding of the connection. This sliding is stopped when dowel action of the longitudinal bars is activated. This dowel action combined with shear-friction mechanism resists the lateral load after initial sliding. Therefore, rebar hinges often do not satisfy the code requirement of a “capacity protected member”. Test results by Mehraein and Saiidi (2015) illustrated that the seismic performance of the base ABC rebar hinges is emulative of CIP connections. Consequently, a seismic
performance rating of 0 is assigned to the base ABC rebar hinges.

2.5.5.2.1. One-Piece Pipe Pin Connection

As an alternative to rebar hinge connections, a one-piece pipe pin connection may be used to improve the hinge seismic performance. Figure 2-4 illustrates a schematic of this connection for CIP construction. To construct a one-piece pipe pin, the central element of the rebar hinge is replaced with an infilled pipe embedded in the column and footing crossing through a gap. The tensile forces produced as a result of overturning effects in multi-column bents and prying action at the interface are conveyed to the footing through welded studs on the surface of the pipe. The base shear is resisted by the infilled pipe and friction force at the column-footing interface. The infilled pipe is designed to prevent the shear failure that occurs in rebar hinges. Similar to rebar hinges, the column is placed over a ring plate on top of the footing to provide a circular hinge gap at the interface, which substantially increases the rotational capacity of the connection.

One-piece pipe pins can be incorporated in ABC utilizing pocket details as shown in Fig. 2-5. The pocket may be formed either in the column or footing. In the former version, an infilled pipe is embedded at the bottom of a precast hollow or solid column and extended into a grout or concrete filled pocket in the footing (Fig. 2-5a). The latter is constructed by embedding the pipe in the footing, extending the protruded portion of the pipe into a hollow precast column placed over the ring plate around the pipe, and filling the hollow section with self-consolidating concrete (Fig. 2-5b).

Among the advantages of the one-piece pipe pins are the better expected seismic performance and ease of construction compared to rebar hinges. This connection type is yet to be studied and implemented in practice. The ratings of the non-seismic parameters for this connection type are the same as those of the EPC-PC-B or EPC-PC-T for infilled pipe extending into the footing pocket or cap beam pocket, respectively.

To provide a general understanding of the behavior and response of one-piece pipe pin connections, preliminary analytical studies of the connection are presented in Appendix B.

2.6. Concluding Remarks

The ratings associated to TRL, seismic performance, and non-seismic evaluation parameters determined in Section 2.5 for the five connection types are compiled in Table 2-10 to provide a better insight about how the connections are compared in terms of different evaluation parameters.

Table 2-10 shows that among the various moment and pin connection details, pocket connections and pipe pin connections, respectively, provide the highest ratings, and thus are proposed to be considered as the first alternatives for use in ABC. EPC-PC offer a substantial reduction in construction time relative to CIP construction, seismic performance that is emulative of CIP connections, and significantly higher composite non-seismic parameter rating compared to other ABC moment connections. The TRL rating for this connection type is as high as those of grouted ducts and grouted-sleeve couplers. The use of pipe pin connections to either eliminate or reduce the moment at the column ends introduces many advantages in terms of the technology readiness, construction speed, seismic performance, and non-seismic considerations compared with CIP or ABC rebar hinge connections.
Tables
Table 2-1. Definition of technology readiness levels for seismic ABC

<table>
<thead>
<tr>
<th>TRL</th>
<th>Definition</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concept exists</td>
<td>The connection concept has been developed.</td>
</tr>
<tr>
<td>2</td>
<td>Static strength predictable</td>
<td>The connection type has been either analyzed or tested for static strength.</td>
</tr>
<tr>
<td>3</td>
<td>Low seismic region deployment</td>
<td>The connection type has been successfully deployed in a low seismic region.</td>
</tr>
<tr>
<td>4</td>
<td>Analyzed for seismic loading</td>
<td>The connection type has been analyzed for response to inelastic cyclic loading.</td>
</tr>
<tr>
<td>5</td>
<td>Seismic testing of components</td>
<td>The critical connection components have been tested under inelastic cyclic loading.</td>
</tr>
<tr>
<td>6</td>
<td>Seismic testing of subassemblies</td>
<td>A connection subassembly has been tested under inelastic cyclic loading.</td>
</tr>
<tr>
<td>7</td>
<td>Design and construction guidelines</td>
<td>Seismic design guidelines for the connection type have been formulated and published.</td>
</tr>
<tr>
<td>8</td>
<td>High seismic region deployment</td>
<td>The connection has been used in a bridge constructed in a high seismic region.</td>
</tr>
<tr>
<td>9</td>
<td>Adequate field performance in earthquakes</td>
<td>The connection type has performed adequately during a design-level seismic event in the field.</td>
</tr>
</tbody>
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Table 2-2. Seismic performance scale

<table>
<thead>
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<th>Scale</th>
<th>Definition relative to CIP</th>
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<tr>
<td>+2</td>
<td>Much better</td>
</tr>
<tr>
<td>+1</td>
<td>Slightly Better</td>
</tr>
<tr>
<td>0</td>
<td>Equal</td>
</tr>
<tr>
<td>-1</td>
<td>Slightly worse</td>
</tr>
<tr>
<td>-2</td>
<td>Much worse</td>
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Table 2-3. Non-seismic parameters scale

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<th>Cumulative Rating</th>
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<tr>
<td>+2</td>
<td>Much better</td>
<td></td>
<td></td>
</tr>
<tr>
<td>+1</td>
<td>Slightly Better</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>Equal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-1</td>
<td>Slightly worse</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-2</td>
<td>Much worse</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scale</td>
<td>Definition relative to CIP</td>
<td>Description of Field Work</td>
<td>Construction Risk</td>
</tr>
<tr>
<td>-------</td>
<td>---------------------------</td>
<td>---------------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>+2</td>
<td>Much better</td>
<td>Detailing is simple and can be done by a reduced construction crew with minimum need for large construction equipment.</td>
<td>There is a very high likelihood that a connection will meet the required quality standard, installation cost, and schedule.</td>
</tr>
<tr>
<td>+1</td>
<td>Slightly Better</td>
<td>Detailing is simple, and fabrication and installation of components can be performed by typically skilled construction workers under predictable conditions using conventional construction equipment.</td>
<td>There is a high likelihood that a connection will meet the required quality standard, installation cost, and schedule.</td>
</tr>
<tr>
<td>0</td>
<td>Equal</td>
<td>Detailing is simple but requires attention to fit-up and appropriate use of materials. Reasonably common supervision is required. Fabrication and installation of components might require a specialty contractor or specialized equipment. Most contractors will be able to successfully construct the project.</td>
<td>There is a high likelihood that a connection will meet the required quality standard, but there is a slight risk for not meeting installation cost or schedule.</td>
</tr>
<tr>
<td>-1</td>
<td>Slightly worse</td>
<td>Detailing is somewhat complex, but skilled construction workers can execute the construction. The work, while complex, is not out of the experience range of a skilled crew but might lead to a slow learning curve, with attendant mistakes, for an inexperienced crew. The work might involve specialty contractors or specialized equipment. Close control will be needed to ensure appropriate quality and final acceptance.</td>
<td>There is a slight risk that a connection will not meet the required quality standard without repairs after initial construction, and there is a moderate risk for not meeting installation cost or schedule.</td>
</tr>
<tr>
<td>-2</td>
<td>Much worse</td>
<td>Detailing is complex and skilled construction workers under close supervision will be required to properly execute the construction. Specialty contractors or specialized equipment will most likely be required for installation work. Tolerances may be close, materials may be difficult to use in the construction, and tight controls over the work must be worked out in advance and specific to the particular project. Mock-ups would typically be beneficial and potentially required. Only the most experienced contractors will be successful with execution of the work.</td>
<td>There is a moderate risk that some repairs may be required after initial construction to satisfy the acceptance criteria for the work, and there is a high risk for not meeting installation cost or schedule.</td>
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Table 2-5. TRL evaluation for grouted-duct connections

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<tr>
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<tr>
<td>4</td>
<td>Analyzed for seismic loading ✓</td>
</tr>
<tr>
<td>5</td>
<td>Seismic testing of components ✓</td>
</tr>
<tr>
<td>6</td>
<td>Seismic testing of subassemblies ✓</td>
</tr>
<tr>
<td>7</td>
<td>Design and construction guidelines ✓</td>
</tr>
<tr>
<td>8</td>
<td>High seismic region deployment ✓</td>
</tr>
<tr>
<td>9</td>
<td>Adequate performance in earthquake ✗</td>
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</table>

Rating 8

Table 2-6. TRL evaluation for mechanical bar couplers

<table>
<thead>
<tr>
<th>TRL</th>
<th>Description</th>
<th>Coupler Type</th>
<th>Bar-Grip</th>
<th>Grouted-Sleeve</th>
<th>Headed-Bar</th>
<th>Shear-Screw</th>
<th>Threaded</th>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>Static strength predictable ✓ ✓ ✓ ✓ ✓ ✓</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Low seismic region deployment ✗ ✓ ✗ ✗ ✗ ✗</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Analyzed for seismic loading ✗ ✓ ✓ ✗ ✓ ✓</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>8</td>
<td>High seismic region deployment ✗ ✓ ✓ ✗ ✓ ✓</td>
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Rating 4 8 6 4 5

Table 2-7. TRL evaluation for pocket connections

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</tr>
<tr>
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<td></td>
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<td>3</td>
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<td>Design and construction guidelines ✗ ✓ ✓ ✓</td>
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Rating 2 6 8 8
Table 2-8. TRL evaluation for pipe pin connections

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<tr>
<td>2</td>
<td>Static strength predictable</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>3</td>
<td>Low seismic region deployment</td>
<td>✗</td>
<td>✗</td>
</tr>
<tr>
<td>4</td>
<td>Analyzed for seismic loading</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>5</td>
<td>Seismic testing of components</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>6</td>
<td>Seismic testing of subassemblies</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>7</td>
<td>Design and construction guidelines</td>
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<td>✓</td>
</tr>
<tr>
<td>8</td>
<td>High seismic region deployment</td>
<td>✗</td>
<td>✓</td>
</tr>
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<td>9</td>
<td>Adequate performance in earthquake</td>
<td>✗</td>
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Rating: 6

Table 2-9. TRL evaluation for rebar hinge connection

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<td>✓</td>
</tr>
<tr>
<td>2</td>
<td>Static strength predictable</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>3</td>
<td>Low seismic region deployment</td>
<td>✗</td>
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<tr>
<td>4</td>
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<td>✓</td>
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<td>Seismic testing of components</td>
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<td>Seismic testing of subassemblies</td>
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Rating: 6 5
Table 2-10. Comparison of different connection types in terms of TRL, seismic performance, and non-seismic parameters

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<tr>
<th>Connection Type</th>
<th>Evaluation Parameters</th>
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<td>TRL</td>
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<td>Time Saving</td>
<td>Initial Cost</td>
<td>Construction Risk</td>
<td>Inspectability</td>
<td>Reparability</td>
<td>Durability</td>
<td>Cumulative Rating</td>
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<tr>
<td>Headed-Bar Coupler</td>
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<td>+2</td>
<td>-1</td>
<td>-2</td>
<td>0</td>
<td>+1</td>
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<td>+2</td>
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<td>-2</td>
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<td>0</td>
<td>-1</td>
</tr>
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<tr>
<td>One-Piece Pipe Pin</td>
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</table>

* Similar to ELB-PC, mechanical bar couplers, or grouted ducts

* Similar to EPC-PC (B or T) depending on the pocket location (footing or column)
Figures
Figure 2-1. Rebar hinge connection alternatives for use in ABC (a) footing pocket detail; (b) column pocket detail
Figure 2-2. Rebar hinge connection alternatives for use in ABC (a) footing mechanical couplers; (b) column mechanical couplers

Figure 2-3. ABC rebar hinge connection using grouted ducts
Figure 2-4. Schematic of one-piece pipe pin connection

Figure 2-5. One-piece pipe pins alternatives for use in ABC (a) footing pocket detail; (b) column pocket detail
Appendix A

Alternatives for Multi-column Bents with Precast Connections and Superstructure-Cap Beam-Column Precast Connections
This appendix presents sketches of different alternatives for precast multi-column bents with non-integral drop caps, column-inverted-T cap precast connections, and column-integral drop cap precast connections.

The sketches are for three categories of the precast multi-column bents: (1) bents with moment connection at the column top and hinge connection at the column base, (2) bents with hinge connection at the column top and moment connection at the column base, and (3) bents with column moment connection at both ends. Different combinations of connections for each category are listed in a separate table (Tables A-1 through A-3). Each bent is labeled with a letter and a number referring to the top connection type, followed by a letter and a number referring to the bottom connections type.

Depending on the column-cap beam connection (hinge vs. moment connection), different alternatives for column-inverted-T cap connections and column-integral drop cap connections are introduced and listed in separate tables (Tables A-4 through A-7). Each connection is labeled with an acronym representing the connection type (ITC for inverted-T cap connection and IDC for integral drop cap connection), followed by a letter and a number referring to the top connections type.

The nomenclature system of the hinge and moment connections was identical among all the multi-column bent categories and superstructure-cap beam-column connections, as listed in the following:

H1: Hinge connection using pipe pin  
H2: Hinge connection using ELB-PC (extended longitudinal bar pocket connection)  
H3: Hinge connection using grouted ducts  
H4: Hinge connection using mechanical couplers  
M1: Moment connection using EPC-PC (extended precast column pocket connection)  
M2: Moment connection using ELB-PC  
M3: Moment connection using grouted ducts  
M4: Moment connection using mechanical couplers
### A.1. Alternatives for Precast Multi-column Bents with Non-integral Drop Caps

Table A-1. Alternatives for precast multi-column bent with moment connection at the top and hinge connection at the bottom of columns

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Top Moment Connection</th>
<th></th>
<th></th>
<th>Bottom Hinge Connection</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pocket EPC-PC</td>
<td>Pocket ELB-PC</td>
<td>Grouted Duct</td>
<td>Mechanical Coupler</td>
<td>Pipe Pin</td>
<td>Pocket ELB-PC</td>
</tr>
<tr>
<td>M1-H1</td>
<td>✓</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M1-H2</td>
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<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M1-H3</td>
<td>✓</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M1-H4</td>
<td>✓</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M2-H1</td>
<td>x</td>
<td>✓</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M2-H2</td>
<td>x</td>
<td>✓</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M2-H3</td>
<td>x</td>
<td>✓</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M3-H1</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>M3-H2</td>
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<td>x</td>
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<td>✓</td>
<td>✓</td>
<td>x</td>
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<td>M3-H3</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M3-H4</td>
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<td>x</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
<tr>
<td>M4-H1</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
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<td>x</td>
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<tr>
<td>M4-H3</td>
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<td>x</td>
<td>x</td>
<td>✓</td>
<td>✓</td>
<td>x</td>
</tr>
</tbody>
</table>
Figure A-1. Moment connection at the top and hinge connection at the bottom- alternative M1-H1
Figure A-2. Moment connection at the top and hinge connection at the bottom - alternative M1-H2
Figure A-3. Moment connection at the top and hinge connection at the bottom - alternative M1-H3
Figure A-4. Moment connection at the top and hinge connection at the bottom- alternative M1-H4
Figure A-5. Moment connection at the top and hinge connection at the bottom - alternative M2-H1
Figure A-6. Moment connection at the top and hinge connection at the bottom- alternative M2-H2
Figure A-7. Moment connection at the top and hinge connection at the bottom - alternative M2-H3
Figure A-8. Moment connection at the top and hinge connection at the bottom - alternative M2-H4
Figure A-9. Moment connection at the top and hinge connection at the bottom- alternative M3-H1
Figure A-10. Moment connection at the top and hinge connection at the bottom- alternative M3-H2
Figure A-11. Moment connection at the top and hinge connection at the bottom- alternative M3-H3
Figure A-12. Moment connection at the top and hinge connection at the bottom - alternative M3-H4
Figure A-13. Moment connection at the top and hinge connection at the bottom - alternative M4-H1
Figure A-14. Moment connection at the top and hinge connection at the bottom - alternative M4-H2
Figure A-15. Moment connection at the top and hinge connection at the bottom- alternative M4-H3
Figure A-16. Moment connection at the top and hinge connection at the bottom- alternative M4-H4
Table A-2. Alternatives for precast multi-column bent with moment connection at the bottom and hinge connection at the top of columns

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Top Hinge Connection</th>
<th>Bottom Moment Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pipe Pin</td>
<td>Rebar Hinge</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pocket Grouted Mechanical</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ELB-PC Duct Coupler</td>
</tr>
<tr>
<td>H1-M1</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>H1-M3</td>
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<td>x</td>
</tr>
<tr>
<td>H1-M4</td>
<td>✓</td>
<td>x</td>
</tr>
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<td>H2-M1</td>
<td>x</td>
<td>✓</td>
</tr>
<tr>
<td>H2-M3</td>
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<td>x</td>
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<td>x</td>
<td>✓</td>
</tr>
<tr>
<td>H3-M4</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>H4-M1</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>H4-M3</td>
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<tr>
<td>H4-M4</td>
<td>x</td>
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Figure A-17. Hinge connection at the top and moment connection at the bottom- alternative H1-M1
Figure A-18. Hinge connection at the top and moment connection at the bottom- alternative H1-M3
Figure A-19. Hinge connection at the top and moment connection at the bottom- alternative H1-M4
Figure A-20. Hinge connection at the top and moment connection at the bottom- alternative H2-M1
Figure A-21. Hinge connection at the top and moment connection at the bottom - alternative H2-M3
Figure A-22. Hinge connection at the top and moment connection at the bottom- alternative H2-M4
Figure A-23. Hinge connection at the top and moment connection at the bottom- alternative H3-M1
Figure A-24. Hinge connection at the top and moment connection at the bottom- alternative H3-M3
Figure A-25. Hinge connection at the top and moment connection at the bottom - alternative H3-M4
Figure A-26. Hinge connection at the top and moment connection at the bottom- alternative H4-M1
Figure A-27. Hinge connection at the top and moment connection at the bottom - alternative H4-M3
Figure A-28. Hinge connection at the top and moment connection at the bottom- alternative H4-M4
Table A-3. Alternatives for precast multi-column bent with moment connection at the top and bottom of columns

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Top Moment Connection</th>
<th>Bottom Moment Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pocket</td>
<td>Grouted</td>
</tr>
<tr>
<td></td>
<td>EPC-PC</td>
<td>Duct</td>
</tr>
<tr>
<td>M1-M1</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td>M1-M3</td>
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</tr>
<tr>
<td>M1-M4</td>
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<td>M2-M1</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>M2-M3</td>
<td>×</td>
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<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>M3-M1</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>M3-M3</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>M3-M4</td>
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</tr>
<tr>
<td>M4-M1</td>
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</tr>
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<td>M4-M3</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>M4-M4</td>
<td>×</td>
<td>×</td>
</tr>
</tbody>
</table>
Figure A-29. Moment connection at the top and bottom- alternative M1-M1
Figure A-30. Moment connection at the top and bottom- alternative M1-M3
Figure A-31. Moment connection at the top and bottom- alternative M1-M4
Figure A-32. Moment connection at the top and bottom - alternative M2-M1
Figure A-33. Moment connection at the top and bottom- alternative M2-M3
Figure A-34. Moment connection at the top and bottom- alternative M2-M4
Figure A-35. Moment connection at the top and bottom- alternative M3-M1
Figure A-36. Moment connection at the top and bottom- alternative M3-M3
Figure A-37. Moment connection at the top and bottom - alternative M3-M4
Figure A-38. Moment connection at the top and bottom- alternative M4-M1
Figure A-39. Moment connection at the top and bottom- alternative M4-M3
Figure A-40. Moment connection at the top and bottom- alternative M4-M4
### A.2. Alternatives for Column-Inverted-T Cap Precast Connections

Table A-4. Alternatives for column-inverted-T cap precast moment connection

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Moment Connection Type</th>
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</thead>
<tbody>
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</tr>
<tr>
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<td>EPC-PC</td>
</tr>
<tr>
<td>ITC-M1</td>
<td>✓</td>
</tr>
<tr>
<td>ITC-M2</td>
<td>×</td>
</tr>
<tr>
<td>ITC-M3</td>
<td>×</td>
</tr>
<tr>
<td>ITC-M4</td>
<td>×</td>
</tr>
</tbody>
</table>
Figure A-41. Inverted-T cap with moment connection- alternative ITC-M1

Figure A-42. Inverted-T cap with moment connection- alternative ITC-M2
Figure A-43. Inverted-T cap with moment connection- alternative ITC-M3

Figure A-44. Inverted-T cap with moment connection- alternative ITC-M4
Table A-5. Alternatives for column-inverted-T cap precast hinge connection

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Hinge Connection Type</th>
<th>Pipe Pin</th>
<th>Pocket ELB-PC</th>
<th>Grouted Duct</th>
<th>Mechanical Coupler</th>
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</thead>
<tbody>
<tr>
<td>ITC-H1</td>
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<td>✗</td>
<td>✗</td>
<td>✗</td>
<td>✓</td>
</tr>
<tr>
<td>ITC-H2</td>
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<td>✗</td>
<td>✗</td>
<td>✓</td>
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<td>✓</td>
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</table>
Figure A-45. Inverted-T cap hinge connection- alternative ITC-H1

Figure A-46. Inverted-T cap hinge connection- alternative ITC-H2
Figure A-47. Inverted-T cap hinge connection- alternative ITC-H3

Figure A-48. Inverted-T cap hinge connection- alternative ITC-H4
### A.3. Alternatives for Column-Integral Drop Cap Precast Connections

Table A-6. Alternatives for column-integral drop cap precast moment connection

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Moment Connection Type</th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pocket</td>
<td>Grouted Duct</td>
<td>Mechanical Coupler</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EPC-PC</td>
<td>ELB-PC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IDC-M1</td>
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<td>×</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>IDC-M2</td>
<td>×</td>
<td>✓</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>IDC-M3</td>
<td>×</td>
<td>×</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td>IDC-M4</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>✓</td>
</tr>
</tbody>
</table>
Figure A-49. Integral drop cap with moment connection- alternative IDC-M1

Figure A-50. Integral drop cap with moment connection- alternative IDC-M2
Figure A-51. Integral drop cap with moment connection- alternative IDC-M3

Figure A-52. Integral drop cap with moment connection- alternative IDC-M4
Table A-7. Alternatives for column-integral drop cap precast hinge connection

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Hinge Connection Type</th>
<th>Pipe Pin</th>
<th>Rebar Hinge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pocket</td>
</tr>
<tr>
<td>IDC-H1</td>
<td>✓</td>
<td>✗</td>
<td>✗</td>
</tr>
<tr>
<td>IDC-H2</td>
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<td>✓</td>
<td>✗</td>
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<td>✓</td>
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<tr>
<td>IDC-H4</td>
<td>✗</td>
<td>✗</td>
<td>✗</td>
</tr>
</tbody>
</table>
Figure A-53. Integral drop cap hinge connection- alternative IDC-H1

Figure A-54. Integral drop cap hinge connection- alternative IDC-H2
Figure A-55. Integral drop cap hinge connection- alternative IDC-H3

Figure A-56. Integral drop cap hinge connection- alternative IDC-H4
Appendix B

Preliminary Analytical Studies of One-piece Pipe Pin Connections
B.1. Introduction

When a pipe is partially embedded in a body of concrete and subjected to external lateral loads applied on the protruded part, its failure may be governed by bearing failure of concrete against the pipe in the lateral direction, shear failure of the pipe, or plastic hinging of the pipe. The load-deformation response of concrete surrounding the pipe can be modeled using a series of unidirectional springs perpendicular to the pipe. The pipe itself can be modeled using nonlinear beam elements including shear deformations. Assuming a uniform concrete bearing stress against the pipe, Zaghi and Saiidi (2010) proposed two different models for the load-deformation relationship of shallow and deep concrete springs.

To provide better insight into the response and performance of the proposed one-piece pipe pin connections, a pin was modeled analytically and subjected to increasing level of rotation and its performance was evaluated. OpenSees, Open System for Earthquake Engineering Simulation, was utilized for modeling and analyzing the nonlinear response of the systems. This appendix presents a brief description of the properties and modeling details of the pin, the results of the analytical suites, and a discussion on the sensitivity of results to different parameters.

B.2. Properties of the Sample Model

Details of the pin is shown in Fig. B-1. The geometrical details and material properties of the pipe pin connections tested by Saiidi and Mehrsorouh (2014) were adopted for the one-piece pipe pin modeling. The pipe was of 4-in. (101.6-mm) outer diameter (Dp), 5/8-in. (15.9-mm) thickness, and embedded to a depth of 5Dp into a cast-in-place footing. The column was of 20-in. (508-mm) diameter and placed over a ring plate on top of the footing to form a 1/4-in. (6.4-mm)-thick hinge gap at the interface. The following assumptions were made in the modeling: (1) the embedment lengths of the pipe in the footing and the column are the same, (2) the pipe is filled with a high-strength grout to enhance the shear capacity, and (3) the pipe tensile forces are resisted through 12 shear studs evenly spaced on the surface of the pipe in the footing and the column.

Figure B-1. Details of sample one-piece pipe pin connection (unit: in. [mm]; I.D.=inner diameter; L=length; O.D.=outer diameter; s=spacing; Ø=diameter)
Table B-1 lists the material properties of the steel parts of the pipe pin. The test-day compressive strength of concrete for the column and footing and the filler grout of the pipe were 6.61 and 7.57, and 10.74 ksi (45.6, 52.2, and 74.0 MPa), respectively.

<table>
<thead>
<tr>
<th>Part</th>
<th>$f_y$ (ksi)</th>
<th>$f_u$ (ksi)</th>
<th>$\varepsilon_y$</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Pipe</td>
<td>84.4</td>
<td>102.2</td>
<td>0.0029</td>
<td>0.25</td>
</tr>
<tr>
<td>Ringe Plate</td>
<td>49.5</td>
<td>66.5</td>
<td>0.00171</td>
<td>0.255</td>
</tr>
<tr>
<td>Shear Stud</td>
<td>62.5</td>
<td>75.5</td>
<td>0.00216</td>
<td>0.24</td>
</tr>
</tbody>
</table>

B.3. Details of Modeling

B.3.1. Model Components

Figure B-2 shows the details of the analytical model for the one-piece pipe pin connection. Two types of nonlinear uniaxial springs were used in this model: axial and shear. Axial springs were used to model the bearing pressure of the concrete against the pipe, and shear springs were utilized to account for shear deformations of the pipe. Uniaxial springs were assigned to the model using zero length elements. The infilled pipe was modeled using fiber section nonlinear beam-column elements to account for flexural deformations. The embedded portion of the pipe in the column was not included in the modeling. Instead, the pipe model was extended up to the column-ring plate interface and its rotational degree of freedom (DOF) was constrained to that of the ring plate outer edge using a rigid link. Base rotation is expected to force the column, and subsequently the embedded pipe to pivot on the outer edge of the ring plate. It was assumed that the hinge gap would be sufficiently wide to prevent the hinge gap closure even under large base rotations. Vertical movement of the pipe was constrained at the bottom to account for the effects of the study. The pipe was subjected to increasing levels of rotation through the top node.

Figure B-2. Analytical model for one-piece pipe pin connection
B.3.2. Nonlinear Concrete Bearing Springs

The load-deformation response of concrete against the pipe was replicated using a series of nonlinear springs. Three parameters are required to define the behavior of concrete springs: elastic stiffness, yield force, and ductility. These parameters were calculated using equations proposed by Zaghi and Saiidi (2010) based on the results of push-off experiments. The elastic stiffness of the springs, $k_c^*$, was calculated using Eq. B-1.

$$ k_c^* = \begin{cases} 
1000 \frac{\sqrt[3]{f_c}}{D_p^{2/3}} & \text{(ksi)} \\
893 \frac{\sqrt[3]{f_c}}{D_p^{2/3}} & \text{(MPa)}
\end{cases} \quad \text{(B-1)} $$

in which:

$D_p = \text{Outer diameter of pipe, in (mm)}$

In this equation, $k_c^*$ is in ksi/in (N/mm$^2$/mm). To determine the elastic stiffness of the springs ($k_c$), this value should be multiplied by the projected area corresponding to tributary length of the pipe that the spring represents.

Equation B-2 was utilized to calculate the yield strength of the springs.

$$ f_c^* = \begin{cases} 
\frac{\sqrt{f_c}}{2.4} \left(2.95 - \frac{D_p^{2/3}}{3.35}\right) f_c & \text{(ksi)} \\
\frac{\sqrt{f_c}}{6.4} \left(2.95 - \frac{D_p^{2/3}}{9.85}\right) f_c & \text{(MPa)}
\end{cases} \quad \text{(B-2)} $$

The corresponding yield force of the springs, $F_{ya}$, was found by multiplying $f_c^*$ by the pipe projected area, which is the same as that used for calculation of $k_c$.

Due to the absence of confining spirals around the pipe, 90% of the calculated values from Eq. B-1 and B-2 were used. Two values were proposed for ductility; one for near surface concrete springs (shallow concrete) and the other for deep concrete springs. Ductility of the springs located within $D_p/4$ from the surface was assumed to be 2.2 (Zaghi and Saiidi, 2010). For deeper springs, a ductility of 16 was assumed. Figure B-3a and B-3b show the complete load-deformation relationship of the shallow and deep springs, respectively.

The calculated yield and ultimate capacities and the corresponding deformations are listed in Table B-2. These properties were assigned to the zero length elements that represent the springs using “ElasticMultilinear” uniaxial material in OpenSees. This material defines the nonlinear force-displacement relationship using a multi-linear curve defined by a set of points. Because the pipe is surrounded by concrete, these springs were needed to be defined on both sides of the pipe. Instead, this requirement was satisfied by modeling one spring at different heights. The spring acted in compression regardless of the direction of pipe displacement.
Figure B-3. Load-deformation relationships for (a) shallow concrete springs and (b) deep concrete springs.

Table B-2. Calculated parameters for defining load-deformation response of footing concrete uniaxial springs

<table>
<thead>
<tr>
<th>$D_p$ (in.)</th>
<th>$f_c$ (ksi)</th>
<th>Spacing (in.)</th>
<th>$k^*_c$ (ksi/in.)</th>
<th>$k_c$ (kip/in.)</th>
<th>$f^*_c$ (ksi)</th>
<th>$F_{ya}$ (kip)</th>
<th>$d_{ya}$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>7.57</td>
<td>0.5</td>
<td>1091.88</td>
<td>2183.76</td>
<td>21.49</td>
<td>42.98</td>
<td>0.0197</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>2.5</td>
<td>1091.88</td>
<td>4367.52</td>
<td>21.49</td>
<td>85.95</td>
</tr>
</tbody>
</table>

Note: 1 in.=25.4 mm; 1 kip=4.45 kN; 1 ksi=6.89 MPa

B.3.3. Nonlinear Shear Springs

The yield and ultimate shear capacity of the infilled steel pipe were calculated using Eq. B-6 and B-7, respectively. The first and second term in these equations represent the contribution of the pipe and the infill material, respectively. The yield and ultimate shear capacities of the pipe was defined as the product of the effective shear area and yield or ultimate shear stress (Eq. B-3 and B-4, respectively) calculated based on Von Mises yield criterion (Ford and Alexander 1963). The
The effective shear area of the pipe was found using Eq. B-5 (Hoogenboom and Spaan 2005). The coefficients of the second terms were determined from the results of the push-off tests by Saiidi and Zaghi (2010).

\[ \tau_y = \frac{f_y}{\sqrt{3}} \quad \text{(B-3)} \]

\[ \tau_u = \frac{f_u}{\sqrt{3}} \quad \text{(B-4)} \]

\[ A_c = \frac{A_v}{2 + t/r} \quad \text{(B-5)} \]

\[ V_y = \frac{A_v f_c}{\sqrt{3} (2 + t/r)} \left[ 0.47A_c \sqrt{f_c} \right] \text{ksi} \]

\[ 1.23A_c \sqrt{f_c} \text{MPa} \quad \text{(B-6)} \]

\[ V_u = \frac{A_v f_u}{\sqrt{3} (2 + t/r)} \left[ 0.93A_c \sqrt{f_c} \right] \text{ksi} \]

\[ 2.47A_c \sqrt{f_c} \text{MPa} \quad \text{(B-7)} \]

where:

- \( A_c \) = Cross-sectional area of the pipe filler material, in\(^2\) (mm\(^2\))
- \( A_v \) = Effective shear area of pipe, in\(^2\) (mm\(^2\))
- \( V_u \) = Ultimate shear capacity of infilled pipe, kip (kN)
- \( V_y \) = Yield shear capacity of infilled pipe, kip (kN)
- \( f_u \) = Measured tensile stress for pipe, ksi (MPa)
- \( f_y \) = Measured yield stress for pipe, ksi (MPa)
- \( t \) = Pipe thickness, in (mm)
- \( r \) = Pipe outer radius, in (mm)
- \( \tau_u \) = Ultimate shear stress, ksi (MPa)
- \( \tau_y \) = Yield shear stress, ksi (MPa)

Based on the pure shear tests of infilled pipes, Zaghi and Saiidi (2010) concluded that the associated yield shear deformation of infilled pipes would be \( D_p/20 \) when the shear span was \( D_p/2 \). They also found that the ultimate capacity of infilled pipes in pure shear was achieved at a displacement ductility of eight. The shear load-deformation response of the pipe is shown in Fig. B-4. Table B-3 lists the calculated parameters for defining the response. Similar to concrete axial springs, the shear load-deformation relationship was defined as “Elastic Multilinear” uniaxial material property and assigned to shear spring zero length elements. Shear springs were defined at location of concrete springs, because shear force between these springs (the shear span) remained constant.
Figure B-4. Shear force-deformation relationship for pipe shear springs

Table B-3. Calculated parameters for defining load-deformation response of infilled pipe shear springs

<table>
<thead>
<tr>
<th>$D_p$</th>
<th>$t$</th>
<th>$f_y$</th>
<th>$f_u$</th>
<th>$A_y$</th>
<th>$s_y$</th>
<th>$\tau_y$</th>
<th>$\tau_u$</th>
<th>$V_y$</th>
<th>$V_u$</th>
<th>$d_{ys}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(in.)</td>
<td>(in.)</td>
<td>(ksi)</td>
<td>(ksi)</td>
<td>(in.$^2$)</td>
<td>(in.)</td>
<td>(ksi)</td>
<td>(ksi)</td>
<td>(kip)</td>
<td>(kip)</td>
<td>(in.)</td>
</tr>
<tr>
<td>4.0</td>
<td>0.625</td>
<td>84.4</td>
<td>102.2</td>
<td>2.87</td>
<td>0.50</td>
<td>0.75</td>
<td>59.01</td>
<td>148.79</td>
<td>187.19</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td>0.075</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.1</td>
</tr>
</tbody>
</table>

Note: 1 in.=25.4 mm; 1 in.$^2$=645.2 mm$^2$; 1 kip=4.45 kN; 1 ksi=6.89 MPa; $s_y$=shear span

B.3.4. Material Property

B.3.4.1. Steel

The “Steel02” uniaxial material was used for modeling the pipe steel in OpenSees. This material is constructed based on the uniaxial Giuffre-Menegotto-Pinto steel material model (1973) with isotropic strain hardening. Six parameters are used to define the steel constitutive stress-strain curves: yield stress, modulus of elasticity, strain hardening ratio, and three other parameters that control the transition from elastic to plastic branches. The first three parameters were obtained from the tensile stress test data (Table B-1). Due to a lack of the full stress-strain relationship, default values were used for defining the transition branch (R0=18.5, cR1=0.925, and cR2=0.15). This material is not capable of modeling the steel failure.

B.3.4.2. Pipe Filler Material

The “Concrete01” uniaxial material was used for modeling the pipe filler material (grout). This material is used to construct a uniaxial Kent-Scott-Park concrete material object with no tensile strength. Because the filler material was surrounded by the pipe, it was confined, and hence its properties were determined using Mander’s model (Mander et al. 1988).
B.4. Results of Analytical Modeling

The one-piece pipe pin connection was subjected to incremental rotation-controlled loading up to 0.1-rad at the top node of the pipe. Figure B-5 shows the distribution of the top node rotation associated with yielding at different depths. This figure indicates that the pipe first yielded at 0.25 in. (6.4 mm) above the footing under a rotation of 0.0081 rad. The strain-rotation relationship of the pipe at the critical node is presented in Fig. B-6. The pipe reached the associated ultimate strain of 0.25 in./in. (mm/mm) at a rotation of 0.0997 rad.

![Figure B-5. Yield rotation distribution along pipe embedded length](image1)

![Figure B-6. Strain-rotation relationship of the pipe at 0.25 in. (6.4 mm) above the footing](image2)

The base rotations cause the column to pivot about the ring plate outer edge resulting in the pipe elongation. The resulting tensile force developed in the pipe is resisted by a concentrated compressive force at the ring plate edge due to prying action. These two forces forms a couple that produces the main portion of the base resisting moment. The moment developing within the pipe is added to the moment due to the prying action. The moment-rotation response of the pipe at the
interface is shown in Fig. B-7. The maximum transferred moment to the footing was 3534.5 kip-in. (399 kN-m) under 0.0839 rad rotation.

Figure B-8 shows the axial force-rotation relationship of the pipe. The pipe underwent a maximum tensile force of 569.08 kip (2,531 kN) at a rotation of 0.0839 rad. Although the pipe was not subjected to external axial forces, an extensive level of tensile force was developed in the pipe due to the prying action of the column at the base. The tensile stresses resulting from the aforementioned net tensile force were added to pipe bending stresses.

![Figure B-7. Base moment-rotation response at interface](image)

![Figure B-8. Axial force-rotation response of pipe](image)

**B.5. Parametric Studies**

**B.5.1. Introduction**

Parametric studies on the effect of ring plate diameter and thickness on the response and performance of one-piece pipe pin connections were performed. Table B-4 lists the parameters and
their ranges. All the test parameters and material properties remained the same as those of the reference model except for the parametric study variables. The parameters were changed one at a time.

Table B-4. Variables of parametric study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Variations</th>
<th>No. of Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ring Plate Diameter</td>
<td>(xDc) (in.) (mm)</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>12</td>
</tr>
<tr>
<td>Ring Plate Thickness</td>
<td>1/80</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>1/40</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>3/80</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>1/20</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Note: Dc=column diameter

B.5.2. Ring Plate Diameter

Figures B-9 through B-11 show the effect of ring plate diameter on the yield strain profile, transferred moment to footing, and pipe tensile force, respectively. Smaller diameters of ring plate delayed the first yield of the pipe until slightly larger base rotations were developed (Fig. B-9). The effect of the ring plate diameter on the base moment and pipe tensile force was more significant (Fig. B-10 and B-11, respectively). Smaller diameter of ring plate forced the column to pivot over the ring plate edge at a smaller offset distance from the pipe center line, hence reducing the pipe elongation and the tensile force (Fig. B-11) and smaller moments at the column-foothing interface.

Figure B-9. Effect of ring plate diameter on yiled strain profile (Dc=column diameter and Dr=ring plate diameter)
Figure B-10. Effect of rингe plate diameter on base moment-rotation relationship (Dc=column diameter and Dr=ring plate diameter)

Figure B-11. Effect of rингe plate diameter on pipe tensile force-rotation relationship (Dc=column diameter and Dr=ring plate diameter)

**B.5.3. Ring Plate Thickness**

The ring plate thickness had a minor effect on the yield strain profile, transferred moment to footing, and pipe tensile force as shown in Fig. B-12 through B-14, respectively. Note that the hinge gap closure was not included in the analyses and the resulting additional moment is not accounted for.
Figure B-12. Effect of ring plate thickness on yield strain profile (Dc=column diameter and tr=ring plate thickness)

Figure B-13. Effect of ring plate thickness on base moment-rotation relationship (Dc=column diameter and tr=ring plate thickness)
Figure B-14. Effect of ring plate thickness on pipe tensile force-rotation relationship (Dc=column diameter and tr=ring plate thickness)