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DESIGN AND CONSTRUCTION OF PRECAST BENT CAPS WITH POCKET CONNECTIONS FOR HIGH SEISMIC REGIONS

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Abstract

In conventional cast-in-place reinforced concrete bridge construction, cap beams and their connection to columns are designed to be capacity protected under strong earthquakes. This is because cap beams and their connections maintain structural integrity and are difficult to repair. The same design philosophy is mandatory for precast cap beams that are used in accelerated bridge construction (ABC), particularly in moderate and high seismic zones. One of the key components of ABC is prefabricated reinforced concrete members. The NCHRP report 698 provided a synthesis of different promising ABC connections. Pocket connections were identified as practical means of joining prefabricated columns and pier caps. The AASHTO Scan 11-02 revealed more recent studies about the seismic performance of pocket connections. Nevertheless, research was needed to develop practical and reliable cap beam pocket connections ensuring capacity protected behavior.

A comprehensive literature search was carried out in the present study to compile and interpret data on the seismic performance of cap beams with pocket connections. It was shown through extensive analyses that effects of pockets on the seismic performance of cap beams are negligible for a well-designed bent cap even under the worst-case scenario in which the concrete within the pocket was excluded from the cap beam section. The reason why precast cap beams with pocket connections yielded in some of the test models was identified as inadequate design rather than the pocket effect. Five practical details for precast pocket bent caps were proposed based on the lessons learned from the aforementioned tasks. Subsequently, constructability of these details was assessed. It was found that the alternative in which fully precast columns are inserted into cap pockets will result in 75% reduction in onsite work. The time saving for other details was 42%. Finally, a design guideline as well as examples were developed to facilitate the field deployment of precast bent caps incorporating pocket connections.

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Executive Summary

ES.1 Introduction

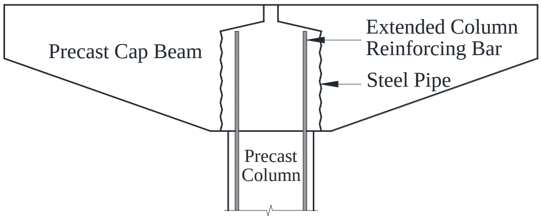
One of the key features of accelerated bridge construction (ABC) is the extensive use of prefabricated bridge elements. Connections of precast elements play a critical role in high seismic regions since the integrity of entire bridge depends on these connections. One of the bridge elements that is appropriate for prefabrication is the bent cap. In conventional reinforced concrete bridge construction, cap beams and their connections are designed to be capacity protected under strong earthquakes since they are difficult to repair. The same design philosophy is mandatory for precast cap beams that are used in ABC, particularly in moderate and high seismic zones. This study was pursued to develop practical and reliable precast bent caps utilizing pocket connections that ensure capacity protected behavior.

ES.2 Objectives

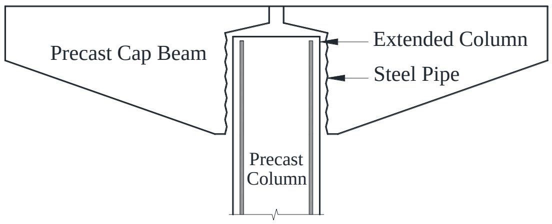
The main objectives of this study were to compile and interpret data on seismic performance of cap beams with pocket connections and to identify behavior, design, detailing, and construction considerations for successful implementation of this category of connections. Five tasks were planned and carried out to achieve these objectives: (1) conducting literature review, (2) determining seismic performance and behavior of pocket connections and cap beams, (3) evaluating constructability of pocket connections, (4) developing design and detailing guidelines for cap beams with a pocket, and (5) demonstrating the guidelines through examples. Highlights of the study and important findings are presented herein.

ES.3 Literature Review

A comprehensive literature search was carried out to investigate seismic performance of columns connected to adjoining members with pocket connections (Fig. ES-1) and a summary of all published and unpublished test data is presented (Table ES-1). The as-built embedment length of bars or precast columns into adjoining members, connection performance, cap beam damage, and the measured yielding of cap beam longitudinal bars are included in the table.



(a) Partially Cast Columns



(b) Fully Precast Columns

(c) Column Embedded in Footing Pocket

Figure ES-1. Pocket Connections

Table ES-1. Summary of Available Test Data on Pocket Connections

Used in	**Reference**	**Emb. Length**	**Connection Performance**	**Cap Beam Performance**	**Yielding in Cap**
Column to Cap Beam	Matsumoto et al. (2001)[(a)]	0.5 column diameter	Plastic hinge formed in column	Minor concrete damage	Not Available
	Restrepo et al. (2011)	1.2 column diameter	27% lower drift capacity compared to cast-in-place, plastic hinge formed in column	Minor radial splitting cracks	Yes, 2.7 times the bar yielding
	Mehrsoroush and Saiidi (2014)	1.2 column diameter	Large drift capacity and large displacement ductility were achieved	No damage of post-tensioned cap beam	No,40% of the yield strain
	Mehraein and Saiidi (2014)	1.0 column diameter	Large drift capacity and large displacement ductility were achieved	Minor damage up 72% of the design level earthquake	No, 70% of the yield strain
Column to Footing	Motaref et al. (2011)	1.5 column diameter	large displacement capacity, no connection damage	Not Applicable	Not Applicable
	Haraldsson et al. (2012)	1.1 column diameter	Similar to cast-in-place, plastic hinge formed in column	Not Applicable	Not Applicable
	Kavianipour and Saiidi (2013)	1.5 column diameter	Minimal spalling of concrete in footing	Not Applicable	Not Applicable
Pile to Cap Beam	Larosche et al. (2014a)	1.3 column diameter	No damage of pile cap was reported	Not Applicable	Not Applicable
	Cukrov and Sanders, 2012	1.2 column diameter	Plastic hinge formed in piles	no apparent damage of cap	No, 50% of the yield strain

[(a)] This was not a "column". It was a RC stub with 4 bars extended to the cap. Was not subjected to cyclic loads that represent earthquakes.

ES.4 Seismic Performance and Behavior of Cap Beam Pocket Connections

Effects of pocket connections were studied using moment-curvature and pushover analyses. First, a full-scale two-column bent was designed based on AASHTO. Then the effects of the pocket were studied on the overall and local behavior of the bent. Table ES-2 presents different scenarios for modeling of a pocket connection. It was found through extensive analyses that the effect of pocket on the seismic performance of cap beams is negligible for a well-designed cap even under worst-case scenarios (SN3 to SN7) in which pocket concrete is excluded from cap beam section resulting in an inverted U-shape section.

Table ES-2. Different Scenarios for Pocket Connection Effects on Reference Bent Behavior

Scenario No	Remarks
SN1	Assign nonlinear material models and nonlinear element to the cap beam with no additional changes compared to the original model used in design in which elastic element was used for the cap beam
SN2	Starting with the analytical model of SN1, bundle cap beam bottom longitudinal reinforcement in corners simulating pocket area
SN3	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1*D* height
SN4	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.1*D* height
SN5	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.2*D* height
SN6	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.3*D* height
SN7	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.4*D* height
SN8	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.5*D* height (Full height of the cap)

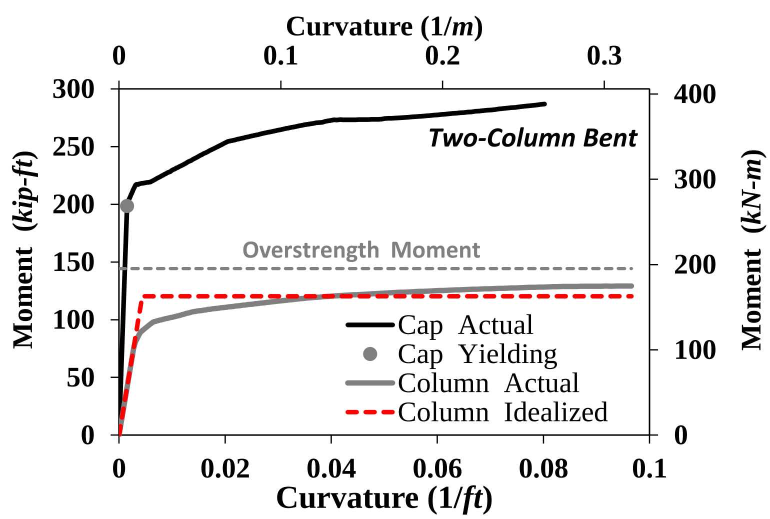


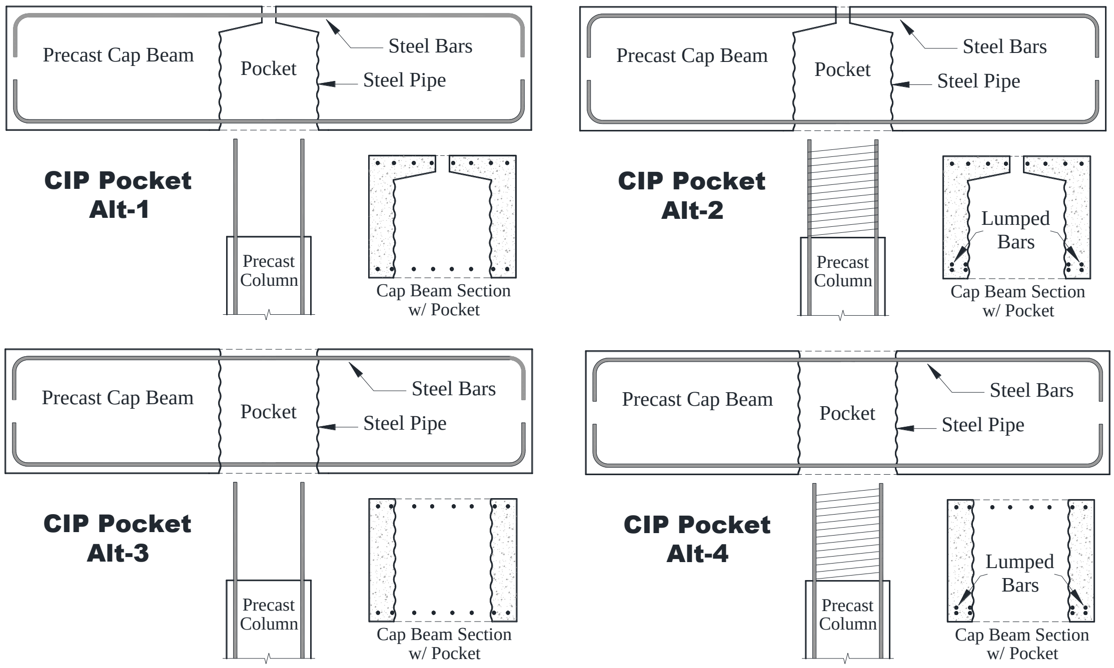
Figure ES-2. Moment-Curvature Relationships for Bent Tested by Mehraein and Saiidi (2014)

Moment-curvature analyses of the test models with pocket connections revealed that cap beams will remain elastic if these elements are designed adequately. Fig. ES-2 shows one sample of the analysis result presented in the report. It can be seen that the yield moment capacity of the precast bent cap was higher than the column overstrength moment satisfying the capacity protected criterion. Post-tensioning of bent caps was found to be a successful method to significantly increase the cap beam yield moment capacity especially when the size of the cap cannot be increased. Furthermore, it was concluded from the analytical results that the reason for cap beam yielding in Restrepo et al. (2011) tests was insufficient design of the cap beams in the test model.

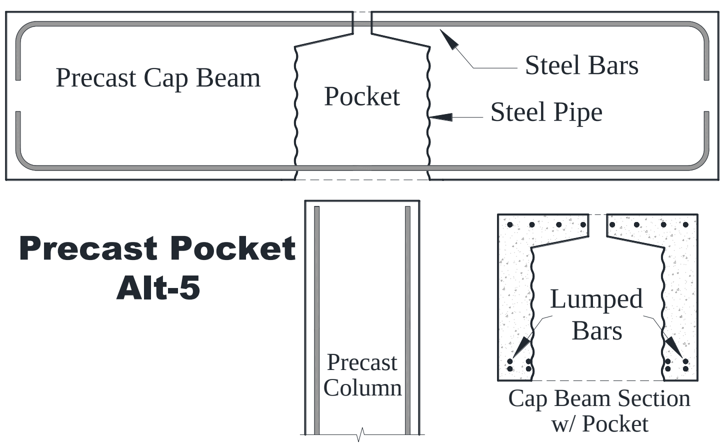
Cap beams should be designed using a legal code such as AASHTO LRFD or AASHTO Guide Specifications to determine the controlling design moment in seismic zones but moment-curvature analyses are recommended to provide insight into the effect of strain hardening and to realistically estimate the cap beam demand to capacity ratio.

ES.5 Constructability of Pocket Connections

Based on the findings of the previous tasks, five practical detailing for cap beam pocket connections were proposed (Fig. ES-3). Constructability of these detailing was discussed and it was pointed out that the size of cap beam incorporating pocket connections will remain the same as conventional cast-in-place cap beam sizes if the AASHTO Guide Specifications are used. The material to fill the pockets, construction tolerance, need for shoring and formwork, and speed of construction were discussed for each alternative.



(a) Cast-in-Place Pocket Connections



(b) Precast Pocket Connection

Figure ES-3. Different Detailing for Bent Cap Pocket Connections

Table ES-3 compares the construction time for each proposed alternative with a cast-in-place bent. The best alternative is Alt-5 in which the construction time is only 25% of that of the cast-in-place bent mainly because there is no need for shoring. In Alt-5, a fully precast column extends into the pocket and the gap between the steel pipe and the column is filled with a fluid grout. The time saving for other alternatives is also significant.

Table ES-3. Construction Time (Day) for Cap Beam Pocket Connections

Construction Step	CIP	Alt-1	Alt-2	Alt-3	Alt-4	Alt-5
Build Shoring/Soffit	4	4	4	4	4	N/A
Set Cap Beam Rebar	2	N/A	N/A	N/A	N/A	N/A
Finish Formwork/Pour Concrete	1	N/A	N/A	N/A	N/A	N/A
Set Shims/Shoring, Sealing and Surveying	N/A	1	1	1	1	1
Set/Level Cap Beam	N/A	0.5	0.5	0.5	0.5	0.5
Pour Pocket Concrete/Grout	N/A	0.5	0.5	0.5	0.5	0.5
Grout Cure Time*	N/A	1	1	1	1	1
Cure Time to 80% (Min 5 Days)*	5	N/A	N/A	N/A	N/A	N/A
Total Construction Time	12	7	7	7	7	3
Total Time Saving (Day)	--	5	5	5	5	9
Total Time Saving (%)	--	42	42	42	42	75

Note: Construction time for CIP is based on Marsh et al. (2011)
* It was assumed that the pocket is filled with grout. If concrete is used, the cure time is 5 days.

ES.6 Design Guideline and Examples

A design guideline (Chapter 4) as well as examples (Chapter 5) were developed to facilitate the field deployment of precast bent caps with pocket connections. The proposed guidelines included both recommendation and commentary to further aid designers. The application of the guidelines was demonstrated though analysis and design of a full-scale, four-column bent incorporating a precast bent cap with pocket connections.

ES.7 Concluding Remarks

Findings from the literature search, evaluations, and analytical studies on precast pocket bent caps led to the following conclusions:

1. Pocket connections can develop full plastic moments in columns when the pocket depth is at least equal to the column largest side dimension (1.0D_c).
2. Columns can be either fully precast to be inserted into pockets or partially cast in which column longitudinal bars are extended into the pockets.
3. Effect of pocket on the seismic performance of bent caps is negligible for a well-designed cap even under the worst-case scenario in which pocket concrete was excluded from cap beam section analysis.
4. In high seismic zones, cap beam must be designed using either the AASHTO LRFD Design Specifications or the AASHTO Guide Specifications for LRFD Seismic Bridge Design to determine the controlling design moment. However, moment-curvature analyses are recommended to provide insight into the effect of strain hardening and to estimate the cap beam capacity realistically.
5. Bent cap post-tensioning can significantly increase the yield capacity of the beam. This is important when the size of cap beam cannot be increased beyond that specified in the guideline.

6. Among the five details proposed for precast pocket cap beams, an alternative in which fully precast columns are inserted into the pockets results in 75% construction time saving mainly because this alternative does not require shoring. Other alternatives result in 42% reduction of onsite activities.
7. The proposed design guidelines are relatively simple and allow designers to choose either force-based or displacement-based bridge design codes.

Table of Content

List of Tables

List of Figures

Chapter 1. Literature Search

1.1 Introduction

Two types of pocket connections are recognized in this project: (1) “cast-in-place” in which the column is prefabricated only up to the bottom of the cap beam with dowels extending into the pocket subsequently filled with grout or concrete from a hole at the top of the cap beam (Fig. 1-1a), and (2) “precast” in which the column is fully precast and is inserted into the cap beam pocket then filled with grout (Fig. 1-1b). Sometimes pocket extends to the top of the cap beam in the former connection type for ease of construction. The latter connection type has been commonly referred to as “member socket connections” in some of the previous studies but this needs to be revisited since the name does not imply its functionality. Therefore, both connection types are generally considered as “pocket connections” in the present study.

A literature search was conducted on the past experimental investigation and field application of both above-mentioned pocket connection types. Connection details and key experimental findings are presented in this task.

1.2 Previous Studies

1.2.1 Matsumoto et al. (2001)

Pullout tests on single-line and double-line grouted pocket systems (Fig. 1-2) were performed by Matsumoto et al. (2001). Several variables such as bar anchorage (straight or headed), bar size (No. 6 [Ø19 *mm*], 8 [Ø25 *mm*], and 11 [Ø36 *mm*]), embedment length (5 to 18 times the bar diameter), number of bars per pocket (single and double bars), and grout type were investigated. Bar pullout and concrete breakout failure were observed in pocket specimens for straight and headed bars, respectively. Design embedment length (L_d) for straight bars in grouted pocket connections was proposed as:

$$L_d = \frac{d_b f_y}{45\sqrt{f'_c}} \qquad (1\text{-}1)$$

where d_b is the bar diameter (*in*.), f_y is the specified yield strength of the bar (*psi*), and f'_c is the specified compressive strength of the bent cap concrete (*psi*). A safety factor of 1.7 was included in this equation accounting for the bar overstrength capacity and the concrete strength reduction factor.

A column connected to a precast cap beam using a double-line pocket system was tested by Matsumoto et al. (2001) in the next phase of their study (Fig. 1-3). The cap beam dimensions were 33×30×144 *in*. (0.84×0.76×3.66 *m*). The column was reinforced longitudinally with twelve No. 9 (Ø29 *mm*) bars and transversely with No. 3 (Ø10 *mm*) spiral spaced at 4 *in*. (102-*mm*) resulting in longitudinal and transverse steel ratio of 1.7% and 0.46%, respectively. Only four of the column longitudinal bars were extended into the cap beam pocket. The column diameter and clear height were 30 *in*. (762 *mm*) and 24 *in*. (610 *mm*), respectively. The column longitudinal bar embedment length into the cap beam was 15 *in*. (381 *mm* or one-half of the column diameter). Two vertical and one horizontal rams were used to obtain load-deflection of connection at service and failure levels under different moment demands. Strain gauges were installed only on the column longitudinal bars, and strain data for bars in the cap beam is not available. Minor damage of concrete in the column and the cap beam was reported at failure (Fig. 1-4) when the column longitudinal bars yielded. Since there was no reference test model, moment-curvature and load-deflection analyses were performed for an analytical model of an assumed cast-in-place (CIP) model and the results were compared with the measured precast test model results. Close correlation was observed between the measured load-deflection and moment-curvature relationships of the column with the pocket connection and the calculated response of the CIP model.

1.2.2 Restrepo et al. (2011)

Restrepo et al. (2011) investigated seismic behavior of a series of precast cap beam to column connections under cyclic loads. Pocket connections were incorporated in two of the test models referred to as "Cap Pocket Full Ductility" (CPFD) and "Cap Pocket Limited Ductility" (CPLD). CPFD and CPLD were designed for high and low seismic regions, respectively. A cast-in-place column model (CIP) was also tested, which was designed according to the 2006 version of AASHTO LRFD Bridge Design Specifications. Table 1-1 presents properties of the specimens and Fig. 1-5 & 1-6 show the cap beam and connection details for all specimens. The drift capacity (and displacement ductility capacity) of CIP, CPFD, and CPLD was reported as 5.9% (μ_d =9.4), 4.3% (μ_d =7.7), and 5.05% (μ_d =9.9), respectively. Even though drift capacity of CPFD and CPLD was respectively 27 and 14% lower than that of CIP, the cap beam longitudinal bars yielded in both pocket specimens while no longitudinal bar yielding was observed in the CIP cap beam. Table 1-2 presents the measured strains of the bars in the cap beam of three test models. The test results showed that the longitudinal bars of the precast cap beams in the extreme layer of reinforcement (bottom layer in the test or top layer in actual cap beam application) yielded at 3.2% drift ratio (corresponding to μ_d =6). Yielding in capacity protected elements such as cap beams is not acceptable.

1.2.3 Mehrsoroush and Saiidi (2014)

Mehrsoroush and Saiidi (2014) tested a 1/3-scale two-column bent in which innovative pipe-pin connections were incorporated at the base of columns, and precast pocket connections were utilized to connect the columns to a post-tensioned (PT) precast cap beam (Fig. 1-7). Figure 1-8 shows photographs of the cap beam during construction. Four longitudinal PVC pipes can be distinguished in this figure, which were subsequently used to pass post-tensioning rods connecting a loading plate to the beam. Each rod was approximately post-tensioned with a 100-*kip* (445-*kN*) force before the test resulting in a total of 400-*kip* (1780-*kN*) compressive load on the cap beam. The cap beam was designed with the expected moment capacity that was 20% larger than the column moment capacity.

The bent was tested under cyclic in-plane loading to failure at a drift ratio of 10.3% and a displacement ductility of 8.7. The testing continued to higher drift ratios. Even under 12% drift ratio cycles, the maximum measured strain of longitudinal bars at the bottom and top of the cap beam was approximately 600 microstrains (30% of the yield strain) and 250 microstrains (12% of the yield strain), respectively. The maximum strain in the spiral around pocket was 800 microstrains (40% of the yield strain).

The post-tensioning force is believed to have contributed only slightly to the satisfactory performance of the cap beam. The estimated compressive strain in the cap beam longitudinal bars due to post-tensioning is 70 microstrains, suggesting that even without the PT force the maximum tensile strain in the cap beam would be substantially less than the yield strain. .

1.2.4 Mehraein and Saiidi (2014)

A shake table test of a 0.27-scale two-column bent was performed by Mehraein and Saiidi (Column-Pile Shaft Pin Connections, 2014). The columns were connected at the base to pile-shafts using pipe pins (Fig. 1-9) and at the top to cap beams incorporating CIP pocket connections. A heavy-duty load-cell was installed between the two cap beams acting as a rigid-link. Figure 1-10 shows a photograph of cap beams during casting. The cap beam was designed based on the column overstrength moment, which was 1.2 times the column plastic moment.

The bent was tested several times under scaled ground motions of the 1994 Northridge earthquake recorded at the Sylmar station with increasing amplitudes. After Run 3, the cap beam was repaired then post-tensioned with a 400-*kips* (1780-*kN*) force since the cap beam to load-cell connection failed during Run 2. Therefore, results up to this run, which was 72% of the design level earthquake, are valid for non-post tensioned cap beams, but afterward the analysis has to include the PT effect.

The test results showed that the column drift demand was 4% in Run 3. Furthermore, the peak measured cap beam longitudinal bar strains were respectively 925 microstrains (42% of the yield strain) and 1550 microstrains (70% of the yield strain) for the bottom and top layers of reinforcement in this run. Thus, no yielding of bars in cap beam was observed up to Run 3. Upon application of PT, the estimated compressive strain in the cap beam longitudinal bars was 150 microstrains. The bent with the post-tensioned cap beams was subsequently tested under stronger motions (85 to 200% of the design level earthquake). The peak measured cap beam longitudinal bar strain at the connection were less than 410 microstrains (18% of the yield strain) confirming capacity protected behavior.

Mehraein and Saiidi tested another bent in which the cap beam was post-tensioned prior to testing. The cap beam detailing and pocket connection were the same as those utilized in the previous test model but the column base to pile shafts connections were two-way hinges with clustered bars. The peak measured cap beam longitudinal strain in the entire test was 150 microstrains (7% of the yield strain) in shake table tests indicating capacity-protected behavior of the cap.

It will be shown in the following chapter that there is a linear relationship between the cap beam post-tensioning force and the cap beam yield moment capacity. Therefore, post-tensioning technique may be used to ensure capacity protected behavior of cap beams when size of cap beam or amount of its reinforcement cannot be increased.

1.2.5 Pocket Connections in Footing or Pile Cap

Pocket connections have been utilized in column to footing or pile to cap connections in a few studies. A summary of these studies is presented in this section for completeness.

Motaref et al. (2011) tested a two-column bent on a shake table in which the columns were connected to the footing using precast pocket connections (Fig. 1-11). Engineered cementitious composite (ECC) was incorporated in the plastic hinge of one of the columns and another column was a concrete-filled fiber reinforced polymer (FRP) tube. Both columns were embedded in the footing with a length of 1.5 column diameters. The embedded part of the column with ECC was constructed using conventional concrete. The precast bent showed large displacement capacity, and no connection damage was observed.

Haraldsson et al. (2012) tested three large-scale columns connected to CIP spread footings using pocket connections. In the first two models, the footing depth (or column embedment length) was approximately equal to the column diameter but the footing depth in the third model was one-half of the column diameter. The column side surface under the column-footing interface was roughened in a sawtooth pattern in all three models. The cyclic tests showed that emulative behavior can be achieved if the column embedment length is at least one column diameter. To demonstrate feasibility in the field, a bridge was built in the State of Washington using this connection type (Fig. 1-12). The column embedment length in the bridge was 1.2 times the column diameter (Khaleghi et al., 2012). The columns were secured then the footing was cast in this project.

A quarter-scale four-span bridge was tested by Kavianipour and Saiidi (2013) on shake tables. Three, two-column bents were constructed with concrete-filled fiber reinforced polymer tubes (CFFTs). The columns in one of three bents were connected to the footing using precast pocket connections with a column embedment length of 1.5 times of the column diameter (Fig. 1-13). The test results showed that full moment response can be expected from these connections making them suitable for high seismic regions.

Two post-tensioned piles with square cross-section were connected to two cap beams using precast pocket connections and were tested under cyclic loads by Larosche et al. (2014a). The pile embedment length was 1.3 times the side dimension of the pile. Satisfactory performance was reported. A full-scale three-pile bent specimen was subsequently tested by Larosche et al. (2014b) (also see Cukrov and Sanders, 2012) in which connection of the piles to a cap beam was provided by pocket systems (Fig. 1-14). The embedment length of piles into the cap beam was 1.2 times the side dimension of piles.

The specimen was tested under displacement-controlled loads simulating the bent cap displacements under the 1992 Landers earthquake recorded at Joshua Tree station. The test results showed that the peak measured longitudinal bars of the cap beam during three times of the original motion was less than one-half of the yield strain. The peak strain was measured in a longitudinal bar of the cap beam top layer reinforcement above Pile C (Fig. 1-14). Therefore, the cap beam performance was satisfactory.

1.3 Field Application

Pocket connections have been used in a few non- and low-seismic states to connect precast cap beams to columns. The Texas Department of Transportation has utilized precast cap beams in several projects. In fact, Texas was the first state to use prefabricated bent caps in the United States (Roddenberry and Servos, 2012). Figure 1-15 shows two projects in which pocket connections were incorporated. The connection detailing shown in the photographs is not appropriate for seismic regions. Other states that used pocket systems in column to cap beam connections are Florida, Iowa, and Minnesota (Marsh et al., 2011). Connection details used in these states are shown in Fig. 1-16.

1.4 Summary

A summary of all published and unpublished test data regarding pocket connections is presented in Table 1-3. The as-built embedment length of bars or precast columns into adjoining members, connection performance, cap beam damage, and the measured yielding of cap beam longitudinal bars were presented.

Chapter 2. Seismic Performance of Cap Beams Incorporating Pocket Connections

2.1 Introduction

A summary of available experimental studies on the seismic performance of pocket connections and cap beams with these connections was presented in previous sections. The measured data for six specimens was reviewed and two cap beams were found to yield and violate capacity protected requirement. AASHTO design procedure regarding cap beams is briefly reviewed in this section and analyses are performed to evaluate design adequacy of previous test models.

2.2 AASHTO Cap Beam Design Philosophy

AASHTO generally allows two methods for seismic design of bridges: force-based design and displacement-based design. The AASHTO LRFD Bridge Design Specification (2013) is based on the force-based design philosophy, but the AASHTO Guide Specifications for LRFD Seismic Bridge Design (2014) presents procedures for displacement-based design of bridges. Cap beams can be designed based on either method but linear-elastic behavior must be guaranteed during earthquakes regardless of the design methodology. Unreduced seismic forces in extreme load combinations are utilized for cap beam design in the forced-based method, whereas for displacement-based design an overstrength factor (usually 1.2 for concrete members) is applied to the plastic moment of columns and used in cap beam design. The intention of using unreduced seismic forces or increased transferred moments to cap beams is to ensure linear-elastic behavior of cap beams, which are considered to be "capacity protected" members. For ABC applications, cap beams should also remain elastic even though different detailing and modified reinforcement arrangement are expected.

2.3 Effect of Pocket Connection on Cap Beam Behavior

Design and construction of cap beams with pocket connections are different from cast-in-place cap beams because of the pockets. Longitudinal reinforcement of the beam can be clustered beside pocket for ease of construction or can pass through the pocket, which is more difficult to construct compared to the former method. Furthermore, it not certain if concrete in the pocket region fully contributes to the cap beam capacity. Effects of these parameters are studied in this section using moment-curvature and pushover analyses. First, a full-scale two-column bent was designed based on AASHTO then effects of the pocket are studied on the overall and local behavior of the bent. Second, the cap beam test models from the available literature are evaluated and reasons for meeting or violating the capacity protected limitation are presented.

2.3.1 Reference Bent Design

A two-column bridge bent was designed based on AASHTO Guide Specification (2014) for a target displacement ductility of 7.5. Note that the AASHTO LRFD Bridge Design Specification (2013) has to be used for initial design of cap beams, which includes frame action in calculating the design forces. Fig. 2-1 shows the bent detailing and Table 2-1 presents a summary of design considerations. Static and initial pushover analyses were performed by SAP2000 (2014) but OpenSees (2014) was used for further nonlinear analyses due to its versatile material models and elements as well as ease of modeling of cap beam sections with or without pockets. The OpenSees modeling method of the reference bent is summarized in Table 2-2 and pushover analysis results are shown in Fig. 2-2. The yield lateral force and drift ratio of the bent were 303 *kips* (1348 *kN*) and 0.46%, respectively, and the effective yield force and drift ratio were 380 *kips* (1691 *kN*) and 0.58%, respectively. The drift capacity of the bent was 4.5%, limited by the crushing of columns core concrete. The displacement ductility capacity of the bent was 7.7. The cap beam design forces were governed by the load combinations from AASHTO LRFD as presented in Table 2-2. The overstrength plastic moment ($1.2M_p$) was 67% of the yield moment of the cap beam (Fig. 2-3) ensuring elastic behavior of the cap.

2.3.2 Effect of Pocket Connection on Reference Bent Behavior

To investigate effects of cap beam pocket connections (Fig. 2-4) on moment-curvature and pushover relationships of the reference bent, eight scenarios were considered (Table 2-3). Figure 2-5 illustrates the cap beam section with pocket for each scenario. It is worth noting that the reference bent was design using an elastic element for the cap beam.

In the first scenario (SN1), a nonlinear fiber-section was assigned to the cap beam utilizing a distributed plasticity force-based element. Five integration points were used for overhang elements (axes A-B and C-D) and seven integration points were utilized for the cap beam (axis B-C). This was done to place the integration points close to the edge of pockets. The integration points for entire cap are marked in circles in Fig. 2-5 with solid circles indicating the cap beam sections with pocket that was used in SN2 to SN8. Bottom layer reinforcement of the cap beam in SN2 was bundled in corners simulating a condition in which pocket is accommodated. The third scenario (SN3) was the same as SN2 but the pocket concrete was excluded from cap section resulting in an inverted U-shape section. The pocket size in SN3 was a cylinder with a diameter of 1.0*D* (*D* is the column diameter) and a height of 1.0*D*. SN4 to SN8 are the same as SN3 but the pocket height was increased successively to 1.5*D* in SN8. The modeling method is summarized in Table 2-4.

Moment-curvature analyses were performed for cap beams described for different scenario (Fig. 2-6). Positive moment was assumed when the cap beam bottom layer reinforcement was in tension. End points of the curves was obtained when either steel bar ruptured or concrete core failed in compression (Table 2-4). It was found that for a well-designed cap beam the effect of pocket inside the cap on moment-curvature response is negligible. The value of the first yield moment (either positive or negative) was insensitive to pocket size (Fig. 2-7). This figure also illustrates the column overstrength moment ($1.2M_p$) in dashed line. It can be seen that for a well-designed cap beam the first yield moment of the cap exceeds the column overstrength moment for all scenarios ensuring linear-elastic (capacity protected) behavior for the cap.

Pushover analyses were also carried out to investigate the pocket effects on the seismic performance of the reference bent as well as local response of the cap. Figure 2-8 shows the pushover curves of the bent for different scenarios. It was found that for a well-designed bent, effect of pocket on the overall bent behavior is insignificant. However, as shown in Fig. 2-9, the maximum longitudinal bar tensile strains in the cap beam increased when bars were bundled in the section corners or when the pocket height increased (resulting in less concrete in the inverted U-shape section simulating pocket) (Fig. 2-9). The increase in the cap beam peak tensile longitudinal bar strains (peak of the both top and bottom bars of the cap in both push and pull directions) was 6 *ksi* (41.4 *MPa*) from SN1 to SN8 but the bars remained elastic even when displacements exceeded the ultimate displacement capacity of the bent. Even though the cap beam of the reference bent remained elastic for different scenarios as shown in Fig. 2-9, the cap beam longitudinal reinforcement could yield in a poorly-designed cap because of increase in the stress demand on the cap beam reinforcement due to the pocket effect.

It can be concluded from the moment-curvature and pushover analyses that the most important factor to achieve linear-elastic behavior for cap beams is how the beams are designed. The effect of pocket in the worst-case scenario in which concrete pocket was excluded from the section was insignificant.

2.4 Moment-Curvature Analyses of Test Models

Past studies that incorporated cap beam pocket connections were discussed in previous sections. Moment-curvature analyses of these studies are presented here to help evaluate the cap beam performance.

2.4.1 Restrepo et al. (2011)

Two inverted column-to-cap beam connections were tested by Restrepo et al. (2011) as shown in Fig. 2-10. These test specimens are the only models among all previously tested cap beams in which cap beam steel bars yielded. To understand the reason for the unsatisfactory performance, moment-curvature analyses were performed for the cap beam and column sections utilizing the measured strength of materials reported in the study.

The measured test day compressive strength of concrete was 5620 *psi* (38.7 *MPa*), and the measured yield and ultimate strengths of steel bars were 63.5 *ksi* (437.8 *MPa*) and 99.6 *ksi* (686.7 *MPa*), respectively.

Figure 2-11 shows moment-curvature relationships for two test specimens (CPFD and CPLD). As discusses in previous sections, CPLD was similar to CPFD but longitudinal and transverse reinforcing steel bars of CPLD were reduced compared to CPFD to examine effects of lower ductility suited for low-seismic regions. It can be seen in Fig. 2-11 that the cap beams would remain elastic if the applied cap beam moment were only 1.2 column plastic moment (overstrength moment). However, since the specimens were tested in an inverted-T configuration, the weight of the column and cap beam, the 38 *kips* (169 *kN*) axial load applied to the column, and part of the weight of the horizontal actuator (Fig. 2-10) increased the cap beam moment to the "Total Moment Demand" marked in Fig. 2-11 and listed in Table 2-5. The total unfactored applied moment ($M_{p, column} + M_{axial} + M_{weight}$) was 359.5 *kip-ft* (55.6 *kN-m*) while the cap beam yield moment capacity was 334.6 *kip-ft* (51.8 *kN-m*) and 357.9 *kip-ft* (55.4 *kN-m*) in CPLD and CPFD, respectively. These findings are in line with the test data in which the peak measured strain of cap beam was 2.74 and 1.41 times of the steel bar yield strain in CPLD and CPFD, respectively.

In summary, it can be concluded from the analytical results that the reason for cap beam yielding in Restrepo et al. (2011) tests was insufficient design of the cap beams that did not include the contribution of the element weights and applied load to the cap beam to the moment.

2.4.2 Mehrsoroush and Saiidi (2014)

The measured test day compressive strength of concrete for cap beam and the CIP column for the two-column bent tested by Mehrsoroush and Saiidi (2014) was 7570 *psi* (52.2 *MPa*) and 6610 *psi* (45.6 *MPa*), respectively. The measured yield and ultimate strengths of the reinforcement were 68.3 *ksi* (471.3 *MPa*) and 109.5 *ksi* (754.9 *MPa*), respectively. There was no axial load applied to the specimen to investigate the uplift effect on the pipe-pin connections at the column base. However, 50% of the weight of cap beam and the columns, which is 13.2 *kips* (58.8 *kN*), was applied to the column model.

Moment-curvature relationships for the cap beam and the CIP column are shown in Fig. 2-12. Even though the cap beam post-tensioning effects were ignored in these analyses, the yield moment capacity of the cap was 100% higher than the column overstrength moment ensuring linear-elastic behavior for the cap. As indicated in previous sections, this cap beam remained elastic during the cyclic test, and the peak measured longitudinal reinforcement strains for the cap beam was only 30% of the steel bar yield strain.

A parametric study was conducted for the cap beam presented in this section to investigate post-tensioning effects on the cap beam yield moment capacity. The base model was without any post-tensioning (PT) forces, whereas the PT model was assumed to be subject to a PT force with 20-*kip* (89-*kN*) increments. Figure 2-13 shows the moment-curvature analysis results. The PT force in the test model, which was 400 *kips* (1780 *kN*), increased the first yield moment capacity of the section by more than 60%. Furthermore, it can be seen that there is a linear relationship between the post-tensioning forces and the section yield moment. Therefore, post-tensioning is proposed as an effective method to increase the cap beam yield moment capacity especially when the size of the cap or the amount of longitudinal reinforcement (either evenly distributed or bundled at the corners) of the cap cannot be increased.

2.4.3 Mehraein and Saiidi (2014)

Moment-curvature analysis was performed for BPSA test model (Column-Pile Shaft Pin Connections, 2014). The cap beam detailing of the second specimen tested by Mehraein and Saiidi was the same as BPSA cap beam detailing but the cap beam was post-tensioned prior to the tests. Therefore, only BPSA was studied herein, which was not post-tensioned up to moderate lateral displacements were measure, but was post-tensioned in the subsequent runs to failure. The cap beam and the precast shell was modeled using the measured strength of materials. The measured compressive strength of concrete for the cap beam and the precast shell was 6310 *psi* (43.5 *MPa*) and 6910 *psi* (47.6 *MPa*), respectively. The core column SCC test day compressive strength was 9870 *psi* (68.1 *MPa*). The measured and ultimate strengths of the longitudinal reinforcement were, respectively, 68 *ksi* (468.8 *MPa*) and 92 *ksi* (634.3 *MPa*). Similar to the test model in Mehrsoroush and Saiidi (2014) no axial load was applied to this specimen during the test. However, one-half of the weight of the elements was applied to the column section in analysis, which was 5.3 *kips* (23.5 *kN*).

Figure 2-14 shows the moment-curvature relationships for the cap beam and the column. The cap beam yield moment was 27% higher than the column overstrength moment ensuring linear-elastic behavior. It is worth noting that the cap beam was designed for the overstrength moment (1.2 column plastic moment). It was mentioned that the cap beam was post-tensioned with a 400-*kip* (1779 *kN*) force after Run 3. Figure 2-15 illustrates the cap beam yield moment versus post-tensioning forces. It can be concluded that even without post-tensioning, the cap beam could remain elastic during shake table tests since the overstrength moment was lower than the cap beam yield moment capacity.

2.5 Summary

A short discussion was presented regarding the design philosophy of capacity protected members. Regardless of the design method and incorporation of ABC connections such as pocket connections, the cap beam must remain elastic under severe earthquakes. It was shown in this section that effects of pocket on the seismic performance of cap beam are negligible for a well-design cap even under the worst-case scenario in which pocket concrete was excluded from cap beam section resulting in an inverted U-shape section. Moment-curvature analyses of the test models with pocket connections revealed that cap beams will remain elastic if these elements are designed adequately. In high seismic zones, cap beam can be designed using either AASHTO LRFD or AASHTO Guide Specification to determine the controlling design moment. However, moment-curvature analyses are recommended to provide insight into the effect of strain hardening and to realistically estimate the cap beam capacity.

Chapter 3. Evaluate Constructability of Pocket Connections

3.1 Introduction

As mentioned in previous sections, two types of pocket connections are recognized in this project: (1) "cast-in-place" in which the column is prefabricated only up to the bottom of the cap beam with dowels extending into the pocket, subsequently filled with grout or concrete from an opening at the top of the cap beam, and (2) "precast" in which the column is fully precast and is inserted into the cap beam pocket then filled with grout. There are some variations in detailing of both connection types: (1) the pocket may extend to the top of the cap beam in the cast-in-place connection type for ease of construction, (2) cap beam longitudinal reinforcement may pass through the pocket in the cast-in-place connection type, and (3) corrugated pipe may serve as the main joint confining mechanism in which either the column spiral extended into the pocket or the spiral cage outside of the corrugated pipe can be eliminated. Figure 3-1 illustrates five practical detailing for cap beams with pocket connections and Table 3-1 presents available test data regarding each alternative. Four alternatives are in the category of cast-in-place pocket connection and one is a precast pocket connection. Constructability of these connections is discussed herein.

3.2 Constructability of Cap Beam Pocket Connections

3.2.1 Cast-in-Place Pocket: Alt- 1

Figure 3-1a shows Alt-1 of the cast-in-place cap beam pocket connection. The longitudinal reinforcement of cap beam Alt-1 is distributed across the width of the beam. The column transverse reinforcement in Alt-1 has to be eliminated because of the interference of bottom reinforcement of cap beam passing through the pocket. The design guideline proposed by Restrepo et al. (2011) recommends a relatively thick corrugated steel pipe to compensate for the lack of transverse reinforcement within the pocket.

The Alt-1 cap beam dimensions may be the same as those of conventional cast-in-place cap beams. The AASHTO Guide Specifications (2014, Article 8.13.4) requirement that the width of bent cap shall extend 12 *in*. (300 *mm*) on each side of the column is sufficient to accommodate steel pipes and the cap beam longitudinal reinforcement with minor construction issues. Self-consolidating concrete (SCC) is recommended to fill the pocket to facilitate construction. The inner diameter of the pocket is recommended to be approximately 4 *in*. (100 *mm*) larger than the column diameter for ease of construction and higher construction tolerance especially in multi-column bents. Another advantage of larger diameter pockets is flexibility in design because the size of off-the-shelf steel pipes changes in increments. Therefore there will not be any need to adjust the column diameter or the column clear cover to fit the column bars into the pocket. Proper formwork and sealing are needed to hold the wet concrete in the pocket during casting. This alternative needs shoring to hold cap beams in-place before casting the pocket with concrete or grout.

3.2.2 Cast-in-Place Pocket: Alt- 2

The difference between Alt-2 and Alt-1 (Fig. 3-1a) is that the bottom-layer longitudinal reinforcement of the cap beam is clustered outside the pocket rather than going through the pocket. This allows for the column transverse reinforcement to extend into the pocket. The AASHTO Guide Specifications (2014) requires the cap beam extend by at least 12 *in*. (300 *mm*) beyond the edge of the column. It can be shown that it is possible to accommodate more than 15-in^2 (9700-mm^2) of the longitudinal reinforcement in the 12-*in.* (300-*mm*) width of the extension without violating the design code. It was experimentally and analytically shown in the previous sections that lumping the cap beam longitudinal reinforcement at the corners has insignificant effects on the seismic behavior of cap beams, and capacity protected performance can be guaranteed via proper design. From construction point of view, Alt-2 is more appealing compared to Alt-1 since there is no intersecting reinforcement in the pocket.

Similar to Alt-1, Alt-2 does not require a larger width or a larger depth for the beam compared to cast-in-place connections. The pocket concrete is recommended to be self-consolidating concrete (SCC). The inner diameter of the pocket is recommended to be approximately 4 *in*. (100 *mm*) larger than the column diameter for ease of construction. Proper formwork and sealing are needed to hold the wet concrete in the pocket. This alternative also needs shoring to hold the cap beam in-place before casting the pocket. Since all components are precast, no additional formwork is needed.

3.2.3 Cast-in-Place Pocket: Alt- 3

Cap beam construction can be facilitated using Alt-3 (Fig. 3-1a) in which the pocket is extended to the top surface of the beam. This is the only variation from Alt-1. All construction limitations and recommendation made for Alt-1 are valid for this alternative as well but the pocket can be filled with conventional concrete instead of SCC since there is sufficient access from the top of the beam to vibrate the concrete.

3.2.4 Cast-in-Place Pocket: Alt- 4

Cap beam construction can be facilitated when the pocket is extended to the top surface of the beam and the cap beam longitudinal bars are clustered adjacent to the pocket. These detailing enhancements lead to Alt-4 as shown in Fig. 3-1a. Detailing for Alt-4 is essentially the same as Alt-2 detailing except for the extension of the pocket to the top surface of the cap beam. All construction limitations and suggestions mentioned for Alt-2 are applicable to this alternative as well but the pocket can be filled with conventional concrete in lieu of SCC.

3.2.5 Precast Pocket: Alt- 5

It is possible to minimize on-site casting for pocket connections utilizing full precast columns as shown in Fig. 3-1b (Alt-5). The bottom longitudinal reinforcement of cap beam in Alt-5 has to be clustered adjacent to the pocket to allow for insertion of the precast column. Another advantages of Alt-5 is that no shoring is needed to support the cap beam.

Similar to previous detailing, Alt-5 does not require a larger width or a larger depth for the beam compared to conventional cast-in-place cap beams. The minimum cap beam width specified by AASHTO Guide Specifications (2014), the column diameter plus 24 *in*. (600 *mm*), is sufficient to accommodate steel pipes and the cap beam longitudinal reinforcement. Only fluid fine-aggregate grout should be used to fill the gap between the column and the pocket. The inner diameter of the pocket is recommended to be approximately 4 *in*. (100 *mm*) larger than the column diameter for ease of construction and to provide higher tolerance for multi-columns bents. Proper formwork and sealing are needed to hold grout in the gap during casting. As indicated before, this alternative does not need shoring. Furthermore, no formwork is needed since all components are precast.

3.3 Speed of Construction

All of the proposed alternatives will result in significant reduction of on-site construction time. Marsh et al. (2011) compared the total column-to-cap beam construction time of a three-column bent built with ABC methods with a similar cast-in-place bent (Fig. 3-2). This bent, which represents typical overpasses in Washington State, was used to compare the construction speed of five alternatives proposed in the present study for pocket connections. Table 3-2 presents number of days needed to complete the construction of each cap beam pocket connection alternative as well as cast-in-place bent (CIP). It can be seen that CIP will be completed in 12 days. A pocket connection with onsite casting of the pocket (Alt-1 to Alt-4) will save five days resulting in 42% saving in construction time compared to the CIP bent. The construction time for a pocket connection with precast column extended into the cap beam (Alt-5) is 75% less than that of CIP connections. Therefore, Alt-5 can be built faster than the other cap beam pocket connections mainly because of no need for shoring, which will result in minimal construction time and cost.

3.4 Summary

Five practical detailing for cap beam pocket connections were proposed in this chapter. Constructability of these detailing was discussed and it was mentioned that the size of cap beam incorporating pocket connections will remain the same as conventional cast-in-place cap beam sizes. Material to fill the pockets, constructional tolerance, need for shoring and formwork, and speed of construction were discussed for each alternative. It was found that the best alternative is Alt-5 in which the construction time is only 25% of that of the cast-in-place bent mainly because there is no need for shoring. In Alt-5, a precast column extends into the pocket and the gap between the steel pipe and the column is filled with fluid grout.

Chapter 4. Design and Detailing Guidelines for Bent Cap Pocket Connections

4.1 Introduction

AASHTO Guide Specifications (2014) provides a comprehensive design method and thorough detailing for capacity protected members such as cap beams and joints (Sections 8.9 to 8.13). Furthermore, Restrepo et al. (2011) proposed design and construction guidelines in NCHRP 681 for precast cap beams with pockets to facilitate field deployment. This chapter is dedicated to development of design guidelines for cap beam pocket connections reflecting new detailing and experimental findings reported in recent studies. Both the Guide Specifications and NCHRP 681 were incorporated in the proposed guidelines, which include recommendations (indicated by "R") and commentary (indicated by "C").

4.2 Proposed Guidelines

R1- Cap beams with pocket connections shall be designed in accordance to a legally adopted bridge code.

C1- Bridge components are analyzed and designed according to the AASHTO LRFD (2013) or AASHTO Guide Specifications (2014) regardless of the use of pocket connections since this connection type is emulative of conventional connections. The detailing requirements to accommodate pockets in bent caps are presented in R2 to R10.

R2- The depth of pocket in a cap beam (H_p) (Fig. R-1) shall be at least the greatest of Eq. R-1 through Eq. R-3:

$$H_p \geq 1.25D_c \qquad \text{(R-1)}$$

$$H_p \geq 0.7d_b.f_{ye}/\sqrt{f'_c} \quad [ksi,\ in.] \qquad \text{(R-2)}$$

$$H_p \geq 24d_b \qquad \text{(R-3)}$$

C2- Experimental studies have shown that full column plastic moment can be transferred to the cap beams when the embedment length of column or column longitudinal reinforcement into the pocket is $1.0D_c$. Eq. R-1 was developed based on these findings including a 1.25 safety factor. Matsumoto et al. (2001) proposed design equation Eq. R-2 for embedment length of column longitudinal bars into the cap beam pockets. The minimum development length of unhooked bars in cap beams according to the Caltrans SDC (2013) is calculated by Eq. R-3.

R3- The depth of bent cap (H_{cap}) shall be allowed to be equal to the pocket depth (H_p) when column longitudinal reinforcement is extended outside the precast column segment and is anchored into the pocket (Alt-3 and 4 in Fig. C-1). For fully precast columns, the depth of bent cap (H_{cap}) shall not be less than $1.25H_p$ as shown in Fig. R-1.

C3- When connecting fully precast columns to cap beams with pocket (Alt-5 in Fig. C-1), the depth of bent cap above the pocket should be sufficiently large to avoid concrete cracking above the pocket during lifting the precast cap beam, and to avoid punching failure above the pocket due to the weight of the precast cap beam. Bent cap depth of $1.25H_p$ can be used as initial design height when columns are either fully or partially precast.

R4- The width of bent cap with pocket (B_{cap}) shall extend at least 15 *in*. (380 *mm*) on each side of the column as shown in Fig. R-1. The gap between the column and the pocket edge shall be no less than 2 *in*. (50 *mm*), but shall not exceed 4 *in*. (100 *mm*) when the column is fully precast. In this case, the bent cap web at the pocket shall be at least 12-*in*. (300-*mm*) wide.

C4- The minimum width of a cap beam according to AASHTO Guide Specifications (2014) is the column diameter (or side dimension) plus 24 *in*. (610 *mm*) (Article 8.13.4.1.1). This limitation was used as baseline in the present guide with a 6-*in*. (150-*mm*) increase to accommodate pocket. The minimum proposed bent cap width (D_p+2.5 *ft*) provides sufficient space to lump all cap beam longitudinal reinforcement in the web. The specified gap between the column and the pocket provides sufficient construction tolerance for multi-column bents while ensuring sufficient grout thickness.

R5- The diameter of the opening above the cap beam pocket (D_h) shall be the greater of (a) three times the maximum size of the coarse aggregate of the pocket filler and (b) 4 *in*. (100 *mm*). At least 10% slope shall be provided for the inner edge of the bent cap above pocket as shown in Fig. R-1.

C5- The American Concrete Pumping Association (2011) recommends limiting the maximum size of the coarse aggregate to one-third of the smallest inside diameter of the pump or placing line. A 4-*in*. (100-*mm*) opening provides sufficient access to cast concrete and grout from top of the bent cap.

R6- Pockets shall be constructed with helical, lock-seam, corrugated steel pipes conforming to ASTM A760. The pipe thickness (t_p) shall be at least:

$$t_p = A_{sp}.f_{yh}/(S_h.f_{yp}.cos\theta) \geq 0.06\ in.(1.5\ mm) \qquad \text{(R-4)}$$

C6- According to ASTM A760, 31 sizes are allowed for corrugated steel pipes with inner diameter of 4 *in*. (100 *mm*) to 144 *in*. (3600 *mm*). Furthermore, seven thicknesses are specified from 0.04 *in*. (1.02 *mm*) to 0.168 *in*. (4.27 *mm*). Table C-1 presents diameter and thickness of steel pipes for practical range of column diameters. Equation R-4, proposed by Restrepo et al. (2011), compensates for the lack of column transverse reinforcement inside the pocket, when column dowels are extended into the pocket, and ensures sufficient confinement by the corrugated steel pipe. Nevertheless, extension of column hoops or spirals into the pocket is highly recommended as illustrated for Alt-2, Alt-4, and Alt-5 in Fig. C-1. Alt-5 is easiest to construct and will result in the highest time-saving. The angle between the horizontal axis of the bent cap and the pipe helical corrugation (θ) is always less than 30-deg for pipes presented in Table C-1 according to the ASTM A760 limitations. Therefore, $\theta = 30°$ may be conservatively used for initial design of the pipe resulting in at most 13% thicker pipes.

R7- The cap beam transverse reinforcement (spiral/hoops) around the pocket (Fig. R-1) shall be placed in the lower half of the bent cap. The transverse reinforcement volumetric ratio shall be the same as that of the column transverse reinforcement.

C7- The required transverse reinforcement around the pocket ensures the integrity of the cap beam in the pocket region. Research has shown that only the transverse reinforcement in the lower half of the pocket is effective in providing confinement (Mehrsoroush and Saiidi, 2014).

R8- Bundling of bent cap longitudinal bars shall be allowed per bridge codes. The bent cap longitudinal bars shall not be discontinuous over the bent length. Bent cap longitudinal bar splices in any form shall not be allowed within $1.0D_c$ from the column center line. Clear cover limitations are not required for inner sides of bent cap sections with pocket.

C8- AASHTO LRFD (2013) specifies the reinforcement detailing (e.g. spacing and bundling) in Section 5.10. Minimum clear cover is not necessary for the reinforcement inside the pocket because the pocket is filled with concrete or grout.

R9- Pocket shall be filled with either concrete, self-consolidating concrete, or grout when columns are partially precast. For fully precast columns, the pockets shall be filled with non-shrink, high-flow grout.

C9- For partially precast columns in which pockets are almost empty after placing the bent cap (Alt-1 to Alt-4 in Fig. C-1), concrete, self-consolidating concrete (SCC), or grout can be used to fill the pocket. However, a filler with no need for vibration (e.g. SCC) is preferred. Grout should be fluid when fully precast columns are embedded in the pocket (Alt-5 in Fig. C-1) since the gap is small. Aggregate-based grout should not be used for Alt-5 since this type of grout is less workable than non-aggregate grout.

R10- Spacers shall be installed above the fully precast columns to provide a vertical gap. This gap shall be no less than 2 *in*. (50 *mm*), but shall not exceed 4 *in*. (100 *mm*). These spacers shall not block grout flow into the gap.

C10- The specified gap between the top surface of the fully precast column and the upper part of the cap beam pocket (Alt-5 in Fig, C-1) ensures that the grout will flow through the entire pocket.

4.3 Notation

A_{sp}: Area of one hoop/spiral as transverse reinforcing steel bar (*in.*^{2}, *mm*^{2})
B_{cap}: Bent cap width (*in., mm*)
d_b: Nominal diameter of column longitudinal reinforcing steel bar (*in., mm*)
D_c: Column largest cross sectional dimension (*in., mm*)
D_h: Hole diameter above pocket (*in., mm*)
D_p: Pocket diameter (*in., mm*)
f'_c: Compressive strength of bent cap concrete (*ksi, MPa*)
f_{ye}: Expected yield stress for longitudinal reinforcing steel bar (*ksi, MPa*)
f_{yh}: Nominal yield stress for transverse reinforcing steel bar (*ksi, MPa*)
f_{yp}: Steel pipe yield stress (*ksi, MPa*)
H_{cap}: Depth of cap beam with pocket (*in., mm*)
H_p: Depth of pocket in cap beam (*in., mm*)
S_h: Spacing of transverse hoops or spirals in equivalent CIP joint
t_p: Pipe thickness (*in., mm*)
θ: Angle between the horizontal axis of the bent cap and the pipe helical corrugation or lock seam (*deg*)

Chapter 5. Design Examples for Cap Beam Pocket Connections

5.1 Introduction

A design guideline was presented in the previous chapter to facilitate application of cap beam pocket connections as a viable ABC connection. This chapter is to demonstrate the guidelines through design of a four-column bent connected to a precast cap beam utilizing pocket connections.

5.2. Reference Cast-in-Place Four-Column Bent

Federal Highway Administration (FHWA) developed a comprehensive bridge design example (Wassef et al. 2003) to aid designers with the implementation of the 2002 AASHTO LRFD Bridge Design Specifications. The FHWA example included a two-span bridge with a four-column bent and prestressed concrete girders. Figure 5-1 shows the bridge, bent, and column and cap beam detailing. The specified concrete compressive strength was 3.0 *ksi* and the steel bars were Grade 60.

This cast-in-place bent was utilized in the present study to illustrate the pocket cap beam design guidelines and to show the changes that are needed to convert the cast-in-place bent cap of the AASHTO example to a precast bent cap.

5.3 Precast Four-Column Bent

Cap beams in which fully precast columns are inserted into pockets (Alt-5) results in minimal onsite construction time among the five proposed alternatives. However, design of cap beam in Alt-5 is more involved than the design of others because Alt-5 does not require shoring. Accordingly, this alternative was selected in this section to fully demonstrate the guideline. The cap beam detailing of the reference CIP bent was modified herein to accommodate the pockets and to satisfy the Alt-5 minimum requirements.

5.3.1 Cap Beam Dimensions

The total depth of the cap beam (H_{cap}) should be at least 1.25 times the pocket depth (H_p). H_p is the greater of (1), (2), and (3) as:

$$H_p \geq 1.25D_c = 1.25 \times 42 = 52.5\ in. \tag{1}$$

$$H_p \geq 0.7d_b.\frac{f_{ye}}{\sqrt{f'_c}} = 0.7 \times 1.0 \times \frac{68}{\sqrt{3.0}} = 27.5\ in. \tag{2}$$

$$H_p \geq 24d_b = 24 \times 1.0 = 24.0\ in. \tag{3}$$

Therefore, H_p = 52.5 *in*. thus H_{cap} = 1.25H_p = 65.6 *in*., or 66 *in*. The minimum width of the cap beam (B_{cap}) is the pocket diameter plus 30 *in*. The diameter of a suitable corrugated steel pipe to form the pocket for this column diameter (42-*in*. diameter) is 48-*in*. Thus,

$$B_{cap} \geq 48 + 30 = 78\ in.$$

The gap between the column and the pocket edge is (48-42)/2=3 *in*., which satisfies the gap requirement.

5.3.2 Bent Cap Depth for Lifting and Punching

The bent cap should remain uncracked during lifting and should be sufficiently strong to resist punching forces when the cap beam bears on the columns. Figure 5-2 shows the precast bent cap moment and punching forces during lifting with the configuration shown. The maximum moment in the pocketed area of the cap beam during lifting due to the cap beam self-weight was 116.5 *kip-ft*, using two lift points as shown in the figure. According to the AASHTO (2013, Article 5.4.2.6), the modulus of rupture for concrete is:

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3} = 0.41\ ksi$$

Thus the cracking moment for the pocketed area of the cap beam (an inverted U-shape section) is:

$$M_{cr} = \frac{f_r.I}{y} = \frac{0.41 \times 1064195}{26.55} \times \frac{1}{12} = 1370\ kip - ft > 116.5\ kip - ft$$

where *I* is the inverted U-shape section moment of inertia and *y* is the distance from the neutral axis to the top edge of the section. The cracking moment at other locations exceeds 1370 *kip-ft* because of the larger sections. The possible cracking should also be checked at the point of the maximum moment. Because the maximum moment of 491.7 *kip-ft* is less than 1370 *kip-ft*, it can be concluded by inspection that the cap beam will not be cracked under self-weight during lifting.

The ACI method (ACI 318-14, Article 22.6.5.2) can be used to estimate the permissible punching shear capacity of the cap beam above the pocket as shown below. Note that the upper part of the cap beam in the pocket area essentially behaves as a slab:

$$V_c = min\begin{bmatrix} \emptyset 4\lambda\sqrt{f'_c}b_o d = 0.75 \times 4 \times 1 \times \sqrt{3000} \times 4 \times 47.5 \times 10.3 \times 10^{-3} = 321\ \ kips \\ \emptyset\left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_o d = 0.75\left(2 + \frac{4}{1}\right)\sqrt{3000} \times 4 \times 47.5 \times 10.3 \times 10^{-3} = 482\ \ kips \\ \emptyset\left(2 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f'_c}b_o d = 0.75\left(2 + \frac{20 \times 10.3}{4 \times 42.95}\right)\sqrt{3000} \times 4 \times 47.5 \times 10.3 \times 10^{-3} = 257\ kips \end{bmatrix}$$

where d is the effective cap beam depth above the pocket (13.5-2-0.625-1.128/2=10.3 *in*.) and b_o is the perimeter of the punching shear critical area. The side dimension of the critical section is the side dimension of an equivalent square column (with an area being the same as the circular column area) plus d/2 ($\sqrt{0.25\pi \times 42^2} + 10.3 = 37.22 + 10.3 = 47.5\ in.$). The punching shear force, or column reactions shown in Fig. 5-2, is 75.31 *kips*, which is well below the controlling permissible shear. Overall, the cap beam depth is sufficient to remain uncracked during lifting and to resist the punching forces when it bears on the columns.

5.3.3 Steel Pipe Thickness

The corrugated steel pipe thickness to form the pocket can be estimated using basic properties of the pipe and the adjoining column. According to the AASHTO example, the columns are transversely reinforced with #3 hoops spaced 12 *in*. on center (Fig. 5-1d). Since the current AASHTO LRFD Bridge Design Specifications (2013) requires higher amount of transverse reinforcement for these columns, new columns reinforced with #5 hoops spaced 12 *in*. on center (according to the AASHTO LRFD Bridge Design Specifications 2013, Articles 5.8.2.5 and 5.8.2.7) was utilized for further analysis. The pipe has a yield strength (f_{yp}) of 30 *ksi* and a 20° helical corrugation. The required pipe thickness is:

$$t_p = \frac{A_{sp}.f_{yh}}{S_h.f_{yp}.cos\theta} = \frac{0.31 \times 60}{12 \times 30 \times cos20} = 0.054\ in. \qquad use\ 0.06\ in.$$

The pipe thickness is calculated based on the column transverse reinforcement to allow the application of pocket connections for cases in which the column transverse reinforcement is not extended into the pocket (e.g. Alt-1, Alt-3).

5.3.4 Precast Bent Detailing

Figure 5-3 shows the precast cap beam detailing. Since the precast cap beam is larger than the reference cast-in-place cap beam, the bent should be reanalyzed and the design forces for the cap beam and the columns should be updated and the capacity should be checked.

It was assumed in this example that the reinforcement in the precast cap beam is the same as that of the reference cast-in-place cap beam. A moment-curvature analysis was carried out to evaluate the precast cap beam capacity. Figure 5-4 shows that the precast cap beam yield moment is 50% larger than the column overstrength moment, making the cap beam a capacity protected member. As indicated before, cap beams should be first designed considering all the AASHTO LRFD load combinations. This is followed by seismic performance evaluation using AASHTO Guide Specifications.

Chapter 6. Summary and Conclusions

6.1 Summary

Pocket connections provide a simple, practical method to assemble precast columns and precast cap beams in accelerated bridge construction. Several studies have been performed in recent years on the seismic performance of pocket connections. The purpose of the study presented in this report was to develop design methods based on the findings of the recent research. A comprehensive literature search was carried out to compile and interpret data on the seismic performance of cap beams with pocket connections. An extensive analytical study was conducted to investigate effects of pockets on the seismic performance of cap beams using several scenarios. The reason why precast cap beams with pocket connections yielded in previous test models was identified, then five practical detailing for precast pocket cap beams were proposed based on the lessons learned from previous studies. Subsequently, constructability of these details was discussed. Finally, a design guideline as well as examples were developed to facilitate field deployment of precast bent caps incorporating pocket connections.

6.2 Conclusions

The findings from the literature search, evaluations, and analytical studies on precast pocket bent caps led to the following conclusions:

1. Pocket connections can develop full plastic moments in columns when the pocket depth is greater than the column largest side dimension (D_c).
2. Columns can be either fully precast to be inserted into pockets or partially cast in which column longitudinal bars are extended into the pockets.
3. Effect of pockets on the seismic performance of bent caps is negligible for a well-designed cap even under the worst-case scenario in which pocket concrete is excluded in the cap beam section analysis.
4. In high seismic zones, cap beams must be designed using either AASHTO LRFD or AASHTO Guide Specifications to determine the controlling design moment. However, moment-curvature analyses are recommended to provide insight into the effect of strain hardening and to estimate the cap beam capacity realistically.
5. Post-tensioning of bent caps can significantly increase the yield capacity of the beam. This is important when there are limits on the size of the cap beam.

6. Among five details proposed for precast pocket cap beams, an alternative in which fully precast columns are inserted into the pockets results in 75% reduction of onsite construction time mainly because no shoring is required for this alternative. Other alternatives result in 42% reduction of onsite activities.
7. The proposed design guidelines are relatively simple and allow designers to choose either force-based or displacement-based bridge design codes.

References

1. AASHTO. (2013). “AASHTO LRFD Bridge Design Specification,” Washington, DC, American Association of State Highway and Transportation Officials.
2. AASHTO. (2014). “AASHTO Guide Specifications for LRFD Seismic Bridge Design,” Washington, DC: American Association of State Highway and Transportation Officials.
3. ACI318. (2014). “Building Code Requirements for Structural Concrete,” Farmington Hills, Michigan: American Concrete Institute.
4. American Concrete Pumping Association. (2011) “Guidelines for the Safe Operation of Concrete Pumps,” Ver. 03.11, Lewis Center, OH, 40 pp.
5. ASTM A760. (2015). “Standard Specification for Corrugated Steel Pipe, Metallic-Coated for Sewers and Drains,” West Conshohocken, PA, 15 pp.
6. Brenes, F.J., Wood, S.L. and Kreger, M.E. (2006). “Anchorage Requirements for Grouted Vertical-Duct Connectors in Precast Bent Cap Systems,” FHWA/TX-06/0-4176-1, Center for Transportation Research, University of Texas at Austin.
7. Caltrans. (2013). “Seismic Design Criteria (SDC),” version 1.7. Sacramento, CA, California Department of Transportation.
8. Column-Pile Shaft Pin Connections. (2014). Retrieved Nov 03, 2014, from http://wolfweb.unr.edu/homepage/saiidi/caltrans/pileshaftpin.html.
9. Cukrov, M. and Sanders, D. (2012). “Seismic Performance of Prestressed Pile-To-Bent Cap Connections,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-12-04.
10. Haraldsson, O.S., Janes, T.M., Eberhard, M.O. and and Stanton, J.F. (2012). “Seismic Resistance of Socket Connection between Footing and Precast Column,” *Journal of Bridge Engineering, ASCE*, Vol. 18, No. 9, pp. 910-919.
11. Khaleghi, B., Schultz, E., Seguirant, S., Marsh, L., Haraldsson, O., Eberhard, M. and Stanton, J. (2012). “Accelerated Bridge Construction in Washington State: From research to Practice,” *PCI Journal*, pp 34-49.
12. Larosche, A., Cukrov, M., Sanders, D., and Ziehl, P. (2014b). “Prestressed Pile to Bent Cap Connections: Seismic Performance of a Full-Scale Three-Pile Specimen,” *Journal of Bridge Engineering, ASCE*, Vol. 19, No. 3, 10 pp.
13. Larosche, A., Ziehl, P., ElBatanouny, M., and Caicedo, J. (2014a). “Plain Pile Embedment for Exterior Bent Cap Connections in Seismic Regions,” *Journal of Bridge Engineering, ASCE*, Vol. 19, No. 4, 04013016, 12 pp.
14. Marsh, M.L., Wernli, M., Garrett, B.E., Stanton, J.F., Eberhard, M.O. and Weinert, M.D. (2011). “Application of Accelerated Bridge Construction

Connections in Moderate-to-High Seismic Regions," Washington, D.C.: National Cooperative Highway Research Program (NCHRP) Report No. 698.
15. Matsumoto, E.E., Waggoner, M.C., Sumen, G. and Kreger, M.E. (2001). "Development of a Precast Bent Cap System," Center for Transportation Research, The University of Texas at Austin: FHWA Report No. FHWA/TX-0-1748-2.
16. Mehrsoroush, A. and Saiidi, M.S. (2014). "Experimental and Analytical Seismic Studies of Bridge Piers with Innovative Pipe Pin Column-Footing Connections and Precast Cap Beams," Center For Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-07, 711 pp.
17. Motaref, S., Saiidi, M.S., and Sanders, D. (2011). "Seismic Response of Precast Bridge Columns with Energy Dissipating Joints," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Report No. CCEER-11-01, 760 pp.
18. OpenSees. (2014). "Open System for Earthquake Engineering Simulations," Version 2.4.1, Berkeley, CA. Available online: http://opensees.berkeley.edu.
19. Restrepo, J.I., Tobolski, M.J. and Matsumoto, E.E. (2011). "Development of a Precast Bent Cap System for Seismic Regions," NCHRP Report 681, Washington, D.C.
20. Roddenberry, M.D. and Servos, J., (2012). "Prefabricated/Precast Bridge Elements and Systems (PBES) for Off-System Bridges," Florida Department of Transportation, Report No. FDOT-BDK83-977-13, 97 pp.
21. SAP2000. (2014). "Integrated Structural Analysis and Design," Computers and Structures, Inc., http://www.csiamerica.com/products/sap2000.
22. Wassef, W.G., Smith, C., Clancy, C.M., and Smith, M.J. (2003) "Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary," Federal Highway Administration Report No. FHWA-NHI-04-044, 384 pp.

Tables

Table 1-1. Details of Test Specimens (Restrepo et al., 2011)

Element	Item	CIP	CPFD	CPLD
Column	Diameter	20 *in*. (508 *mm*)	20 *in*. (508 *mm*)	20 *in*. (508 *mm*)
	Length	45 *in*. (1143 *mm*)	46.5 *in*. (1181 *mm*)	46.5 *in*. (1181 *mm*)
	Longitudinal Reinforcement	16 No. 5 (16 Ø16 *mm*) [1.58%]	16 No. 5 (16 Ø16 *mm*) [1.58%]	16 No. 5 (16 Ø16 *mm*) [1.58%]
	Transverse Reinforcement	No. 3 (Ø16 *mm*) at 2 *in*. (51 *mm*)	No. 3 (Ø16 *mm*) at 2 *in*. (51 *mm*)	No. 3 (Ø16 *mm*) at 2 *in*. (51 *mm*)
Bent Cap	Longitudinal Reinforcement	12 No. 5 (12 Ø16 *mm*) [0.65%] at Top & Bot.	12 No. 5 (12 Ø16 *mm*) [0.65%] at Top & Bot.	8 No. 5 (8 Ø16 *mm*) & 2 No. 4 (2 Ø13 *mm*) [0.50%] at Top & Bot.
	Transverse Reinforcement	2-leg No. 3 (Ø10 *mm*) stirrups at 6 *in*. (152 *mm*)	2-leg No. 3 (Ø10 *mm*) stirrups at 6 *in*. (152 *mm*)	2-leg No. 3 (Ø10 *mm*) stirrups at 8 *in*. (203 *mm*)
Joint	Helical Pipe	None	Diameter: 18 *in*. (457 *mm*) Thickness: 0.065 *in*. (1.65 *mm*)	Diameter: 18 *in*. (457 *mm*) Thickness: 0.065 *in*. (1.65 *mm*)
	Vertical Stirrups, Horizontal Cross Tie	External to Joint Only	External to Joint Only	None
	Other Reinforcement	Two 2-leg construction stirrups placed in joint	Two 2-leg construction stirrups placed in joint	None

Table 1-2. Measured Strain Cap Beam Bars in CIP, CPFD, and CPLD (Restrepo et al., 2011)

Top Bars		S2		S1		CL		N1		N2	
		Push	Pull	Push	Pull	Push	Pull	Push	Pull	Push	Pull
CIP (LB13)	μ2	-218	118	146	252	–	–	267	-208	–	–
	μ4	-194	141	80	228	–	–	168	-392	–	–
	μ6	-229	116	30	302	–	–	53	-581	–	–
	$\varepsilon_{max}/\varepsilon_y$	-0.10	0.06	0.07	0.14	–	–	0.12	-0.26	–	–
CPFD (LB7)	μ2	–	–	–	–	774	1233	160	-461	72	-166
	μ4	–	–	20	-31	833	1725	144	-472	87	-180
	μ6	–	–	-134	-189	1050	2169	189	-494	75	-167
	$\varepsilon_{max}/\varepsilon_y$	–	–	-0.06	-0.09	0.48	0.99	0.09	-0.23	0.04	-0.08
CPLD (LB7)	μ2	-70	106	682	326	605	235	409	217	–	–
	μ4	-107	82	1207	519	642	344	516	412	–	–
	μ6	-64	163	1365	691	667	292	712	855	–	–
	$\varepsilon_{max}/\varepsilon_y$	-0.05	0.07	0.62	0.32	0.30	0.16	0.32	0.39	–	–

Bottom Bars		S2		S1		CL		N1		N2	
		Push	Pull	Push	Pull	Push	Pull	Push	Pull	Push	Pull
CIP (LB10)	μ2	823	-108	968	68	891	221	259	755	78	535
	μ4	867	-135	875	132	947	334	349	876	59	587
	μ6	890	-164	1,021	157	961	434	449	1,018	35	685
	$\varepsilon_{max}/\varepsilon_y$	0.40	-0.07	0.46	0.07	0.43	0.20	0.20	0.46	0.04	0.31
CPFD (LB13)	μ2	674	-107	585	-20	1,792	525	188	1,138	21	418
	μ4	737	-121	1,315	279	2,357	800	270	1,372	15	448
	μ6	757	-135	1,540	446	3,094	890	343	1,549	–	–
	$\varepsilon_{max}/\varepsilon_y$	0.35	-0.06	0.70	0.20	1.41	0.41	0.16	0.71	0.01	0.20
CPLD (LB13)	μ2	1131	180	1069	799	1145	854	574	580	288	617
	μ4	1112	113	1096	790	1161	866	383	699	312	638
	μ6	1200	239	1133	847	1265	6000	176	868	472	768
	$\varepsilon_{max}/\varepsilon_y$	0.55	0.11	0.52	0.39	0.58	2.74	0.26	0.40	0.22	0.35

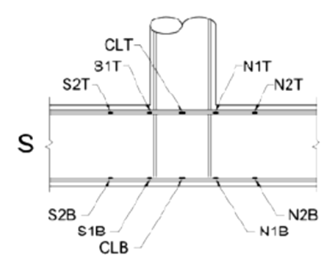


Table 1-3. Summary of Available Test Data on Pocket Connections

Used in	Reference	Emb. Length	Connection Performance	Cap Beam Performance	Yielding in Cap
Column to Cap Beam	Matsumoto et al. (2001)[(a)]	0.5 column diameter	Plastic hinge formed in column	Minor concrete damage	Not Available
	Restrepo et al. (2011)	1.2 column diameter	27% lower drift capacity compared to cast-in-place, plastic hinge formed in column	Minor radial splitting cracks	Yes, 2.7 times the bar yielding
	Mehrsoroush and Saiidi (2014)	1.2 column diameter	Large drift capacity and large displacement ductility were achieved	No damage of post-tensioned cap beam	No,40% of the yield strain
	Mehraein and Saiidi (2014)	1.0 column diameter	Large drift capacity and large displacement ductility were achieved	Minor damage up 72% of the design level earthquake	No, 70% of the yield strain
Column to Footing	Motaref et al. (2011)	1.5 column diameter	large displacement capacity, no connection damage	Not Applicable	Not Applicable
	Haraldsson et al. (2012)	1.1 column diameter	Similar to cast-in-place, plastic hinge formed in column	Not Applicable	Not Applicable
	Kavianipour and Saiidi (2013)	1.5 column diameter	Minimal spalling of concrete in footing	Not Applicable	Not Applicable
Pile to Cap Beam	Larosche et al. (2014a)	1.3 column diameter	No damage of pile cap was reported	Not Applicable	Not Applicable
	Cukrov and Sanders, 2012	1.2 column diameter	Plastic hinge formed in piles	no apparent damage of cap	No, 50% of the yield strain

(a) This was not a “column”. It was a RC stub with 4 bars extended to the cap. Was not subjected to cyclic loads that represent earthquakes.

Table 2-1. Design Parameters for Reference Two-Column Bent

Parameter	**Remarks**
Scale	Full
Column Height	30 *ft* (9.14 *m*) clear
Column Diameter	4 *ft* (1.22 *m*)
Column Long. Reinforcement	22-#9 (22-Ø29 *mm*), ρ_l =1.21%
Column Trans. Reinforcement	#5 (Ø16 *mm*) hoops at 4 *in*. (102 *mm*), ρ_s =0.71%
Cap Beam Length	48 *ft* (14.63 *m*) overall
Cap Dimension	6 *ft* by 6 *ft* (1.82 *m* by 1.82 *m*)
Concrete Strength for all Elements	4000 *psi* (27.58 *MPa*)
Cover Concrete for all Elements	2 *in*. (51 *mm*)
Dead Load excluding cap and columns weights	30.16 *kips*/*ft* (440.1 *kN*/*m*) resulting in 10% axial load index[a]
AASHTO LRFD Consideration and Results[b]	
Bridge Site	Downtown of Los Angeles, USA
Soil Site Class	*D*
Code Version in USGS Design Tool	AASHTO 2009
Design Seismic Spectrum	A_s=0.64, S_{DS}=1.515, S_{DS}=0.772, T_0=0.102 sec, T_s=0.51 sec
Hand Calculated Period of the Bent	1.28 *sec* using cracked stiffness for the columns
First Mode Period of the Bent	SAP2000: 1.34 *sec*; OpenSees: 1.38 *sec*
Earthquake Load Calculation	Response Spectrum Analysis, mass from dead load and elements weight
Response Modification Factor, *R*	5
Base Shear from Response Spectrum Analysis	214.5 *kips* (954 *kN*)
Design Level Bent Displacement	2.04 *in*. (52 *mm*) equivalent to 0.52% drift ratio
Design Load Combinations	$1.25D \pm 1.0EQ$; $0.9D \pm 1.0EQ$
AASHTO Guide Specification Consideration and Results[c]	
Bent Target Displacement Ductility Capacity	7.5
Cap Beam Model	Elastic element for pushover analysis
Bent Failure	15% reduction in lateral strength of the bent caused by either core crushing or bar rupture
Bent First Yield Displacement	1.83 *in*. (46 *mm*) equivalent to 0.46% drift ratio
Bent First Yield Force	303 *kips* (1348 *kN*)
Bent Effective Yield Displacement	2.3 *in*. (58 *mm*) equivalent to 0.58% drift ratio
Bent Effective Yield Force	380 *kips* (1691 *kN*)
Bent Displacement Capacity	17.85 *in*. (453 *mm*) equivalent to 4.5% drift ratio resulting in displacement ductility capacity of 7.75

Note:
[a] Axial Load Index is the ratio of the axial load to the product of the compressive strength of concrete and the column cross section area
[b] Based on AASHTO LRFD Bridge Design Specification (2013)
[c] Based on AASHTO Guide Specifications for LRFD Seismic Bridge Design (2014)

Table 2.2- Modeling Method for Design of Reference Two-Column Bent

General Remarks	
Column Model: Element: *forceBeamColumn* with 5 integration points Section: Fiber section Cover Concrete Discretization: 10 radial by 10 circumferential Cover Concrete Discretization: 30 radial by 10 circumferential $P - \Delta$ effects was included No bond-slip effects	Cap Beam: Element: *Elastic* element with a rigidity based on cap beam actual size
Column Concrete Fibers	
Application: unconfined concrete Type: *Concrete01* f'_{cc}= -4000 *psi* (-27.58 *MPa*) ε_{cc}= -0.002 *in./in.* f'_{cu}= 0.0 *psi* (0.0 *MPa*) ε_{cu}= -0.005 *in./in.*	Application: confined concrete (based on Mander's model) Type: *Concrete04* f'_{cc}= -5260 *psi* (-36.3 *MPa*) ε_{cc}= -0.0037 *in./in.* f'_{cu}= -4629 *psi* (-31.9 *MPa*) ε_{cu}= -0.0147 *in./in.* f_{ct}= 395 *psi* (2.72 *MPa*), based on ACI318-11 E_t= 30663 *psi* (211 *MPa*)
Column Steel Fibers	
Application: All integration point (based on AASHTO Guide Specification) Type: *ReinforcingSteel* f_y= 68.0 *ksi* (468.8 *MPa*) f_{su}= 95.0 *ksi* (665.0 *MPa*) E_s= 29000 *ksi* (63252 *MPa*) E_{sh}= 0.043E_s ε_{sh}= 0.0125 *in./in.* (may use smaller value to converge*) ε_{su}= 0.09 *in./in.*	None

* It was found that the yield plateau of this steel model is source of convergence issue in many cases. Smaller yield plateau (smaller ε_{sh}) compared to AASHTO Guide Spec value may be used.

Table 2-3. Different Scenarios for Pocket Connection Effects on Reference Bent Behavior

Scenario No	Remarks
SN1	Assign nonlinear material models and nonlinear element to the cap beam with no additional changes compared to the original model used in design in which elastic element was used for the cap beam
SN2	Starting with the analytical model of SN1, bundle cap beam bottom longitudinal reinforcement in corners simulating pocket area
SN3	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1*D* height
SN4	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.1*D* height
SN5	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.2*D* height
SN6	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.3*D* height
SN7	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.4*D* height
SN8	Starting with the analytical model of SN2, exclude concrete from pocket area in which pocket is a cylinder with approximately one column diameter (1*D*) and 1.5*D* height (Full height of the cap)

Table 2-4. Modeling Method for Moment-Curvature and Pushover Analyses of Reference Bent

General Remarks	
Column Model: Element: *forceBeamColumn* with 5 integration points Section: Fiber section Cover Concrete Discretization: 10 radial by 10 circumferential Cover Concrete Discretization: 30 radial by 10 circumferential $P - \Delta$ effects was included No bond-slip effects	Cap Beam: Element: three *forceBeamColumn* elements. Overhang elements were modeled with 5 integration points, and the cap beam between the two columns was modeled with 7 integration points. This was done to be able to simulate pocket locations in the cap.
Cap Beam Concrete Fibers	
Application: unconfined concrete Type: *Concrete01* f'_{cc}= -4000 *psi* (-27.58 *MPa*) ε_{cc}= -0.002 *in./in.* f'_{cu}= 0.0 *psi* (0.0 *MPa*) ε_{cu}= -0.005 *in./in.*	Application: confined concrete (based on Mander's model) Type: *Concrete04* f'_{cc}= -4520 *psi* (-31.1 *MPa*) ε_{cc}= -0.0054 *in./in.* f'_{cu}= -3435 *psi* (-23.7 *MPa*) ε_{cu}= -0.0116 *in./in.* f_{ct}= 395 *psi* (2.72 *MPa*), based on ACI318-11 E_t= 30663 *psi* (211 *MPa*)
Cap Beam Steel Fibers	
Application: All integration point (based on AASHTO Guide Specification) Type: *ReinforcingSteel* f_y= 68.0 *ksi* (468.8 *MPa*) f_{su}= 95.0 *ksi* (665.0 *MPa*) E_s= 29000 *ksi* (63252 *MPa*) E_{sh}= 0.043E_s ε_{sh}= 0.0125 *in./in.* ε_{su}= 0.09 *in./in.*	None

Note:
Column modeling method was presented in Table 2-2

Table 2-5. Design Moment for Cap Beams in Test Models of Restrepo et al. (2011)

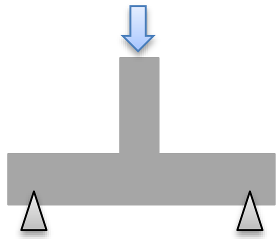
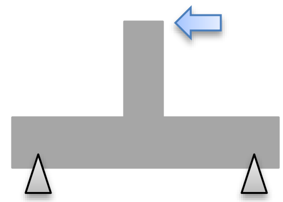
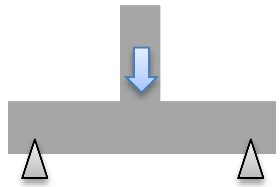
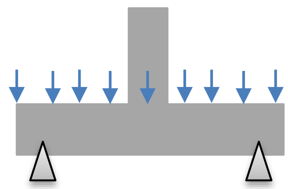
 Column Axial Load= 38 *kips* Cap Beam Peak Moment, M_{axial}=PL/4=38*10/4=95 *kip-ft*	 Column Plastic Moment, M_p= 241.4 *kip-ft*
 Column Weight= 2 *kips* 50% of the Actuator Weight (assumed)= 4 *kips* Cap Beam Peak Moment, $M_{col\text{-}w}$=PL/4=6*10/4=15 *kip-ft*	 Cap Weight= 0.65 *kips/ft* Cap Beam Peak Moment, $M_{cap\text{-}w}$= $wL^2/8$=8.1 *kip-ft*
Unfactored Deign Moment for Cap Beam= $M_{axial} + M_p + M_{col\text{-}w} + M_{cap\text{-}w}$ =359.5 *kip-ft*	

Table 3-1. Practical Detailing for Cap Beam Pocket Connections

Pocket	Alternative	Description	References
Cast-in-Place	Alt-1	Pocket is cast in-place with concrete/grout, cap beam longitudinal reinforcement is distributed across the width of the beam, no spiral for column in the pocket	No testing
	Alt-2	Pocket is cast in-place with concrete/grout, cap beam bottom-layer longitudinal reinforcement is lumped in the web of the inverted U-shape section, continuous spiral for column in the pocket	Mehraein and Saiidi (2014)
	Alt-3	Pocket is cast in-place with concrete/grout, cap beam longitudinal reinforcement is distributed across the width of the beam, no spiral for column in the pocket, pocket is extended to the top of the beam	Restrepo et al. (2011)
	Alt-4	Pocket is cast in-place with concrete/grout, cap beam bottom-layer longitudinal reinforcement is lumped outside the pocket, continuous spiral for column in the pocket, pocket is extended to the top of the beam	No testing
Precast	Alt-5	Gap between the steel pipe and the column in the pocket is cast in-place with grout, cap beam bottom-layer longitudinal reinforcement is clustered outside the pocket	Mehrsoroush and Saiidi (2014)

Table 3-2. Construction Time (Day) for Cap Beam Pocket Connections

Construction Step	CIP	Alt-1	Alt-2	Alt-3	Alt-4	Alt-5
Build Shoring/Soffit	4	4	4	4	4	N/A
Set Cap Beam Rebar	2	N/A	N/A	N/A	N/A	N/A
Finish Formwork/Pour Concrete	1	N/A	N/A	N/A	N/A	N/A
Set Shims/Shoring, Sealing and Surveying	N/A	1	1	1	1	1
Set/Level Cap Beam	N/A	0.5	0.5	0.5	0.5	0.5
Pour Pocket Concrete/Grout	N/A	0.5	0.5	0.5	0.5	0.5
Grout Cure Time*	N/A	1	1	1	1	1
Cure Time to 80% (Min 5 Days)*	5	N/A	N/A	N/A	N/A	N/A
Total Construction Time	12	7	7	7	7	3
Total Time Saving (Day)	--	5	5	5	5	9
Total Time Saving (%)	--	42	42	42	42	75

Note: Construction time for CIP is based on Marsh et al. (2011)

* It was assumed that the pocket is filled with grout. If concrete is used, the cure time is 5 days.

Table C-1. Galvanized Steel Pipe Dimension for Cap Beam Pocket Connections

Inside Diameter, *in*. (*mm*)	Specified Thickness, *in*. (*mm*) [2 2/3” x 1/2” Corrugation]	Specified Thickness, *in*. (*mm*) [3” x 1” or 5” x 1” Corrugation]
36 (900)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51)	N/A
42 (1050)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)	N/A
48 (1200)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)	N/A
54 (1350)	0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
60 (1500)	0.109 (2.77) 0.138 (3.51) 0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
66 (1650)	0.109 (2.77) 0.138 (3.51) 0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
72 (1800)	0.138 (3.51) 0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
78 (1950)	0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
84 (2100)	0.168 (4.27)	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)
90 (2250)	N/A	0.064 (1.63) 0.079 (2.01) 0.109 (2.77) 0.138 (3.51) 0.168 (4.27)

Figures

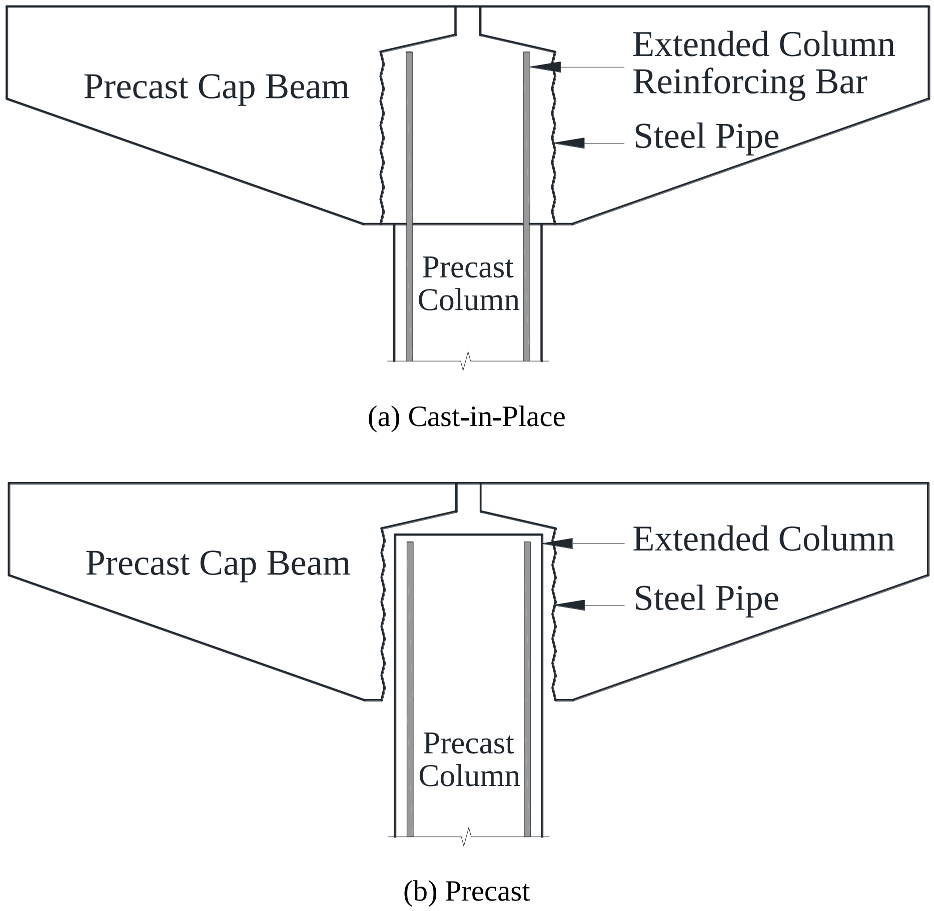
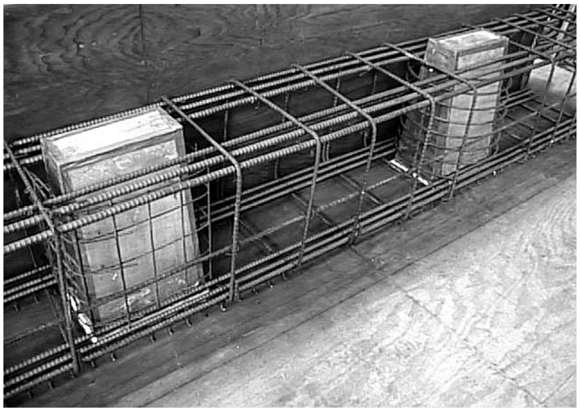
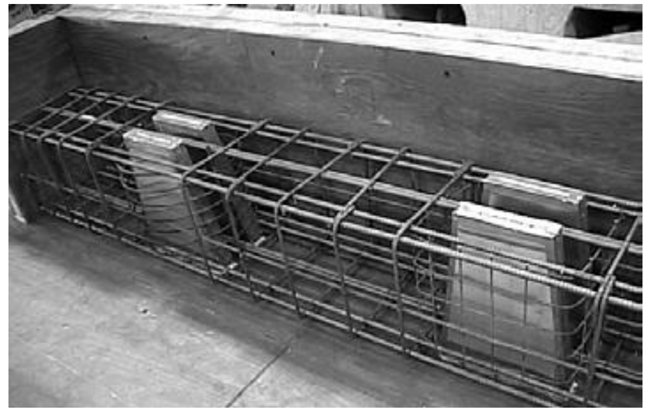


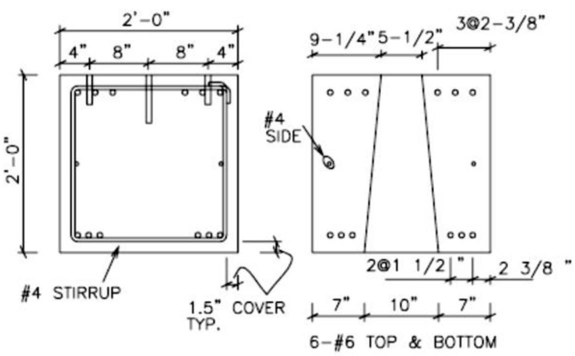
Figure 1-1. Pocket Connections



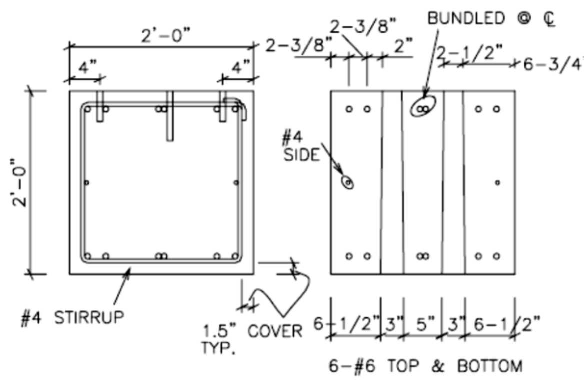
(a) Rebar Cage for Single-Line Pocket



(b) Rebar Cage for Double-Line Pocket

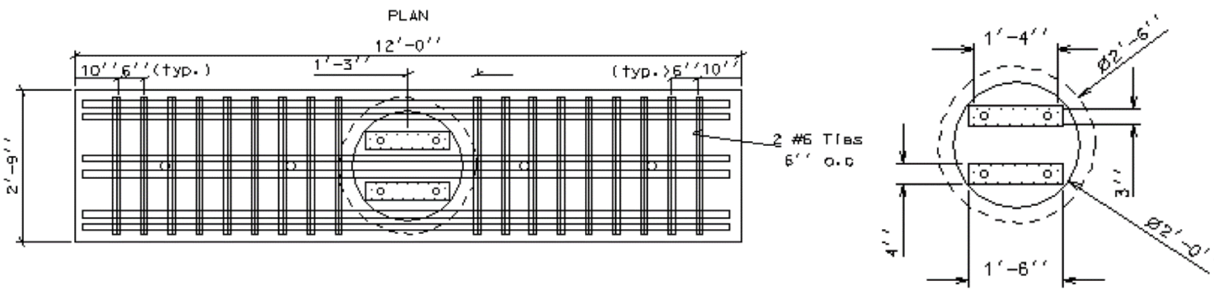


(c) Cap Beam Details for Single-Line Pocket

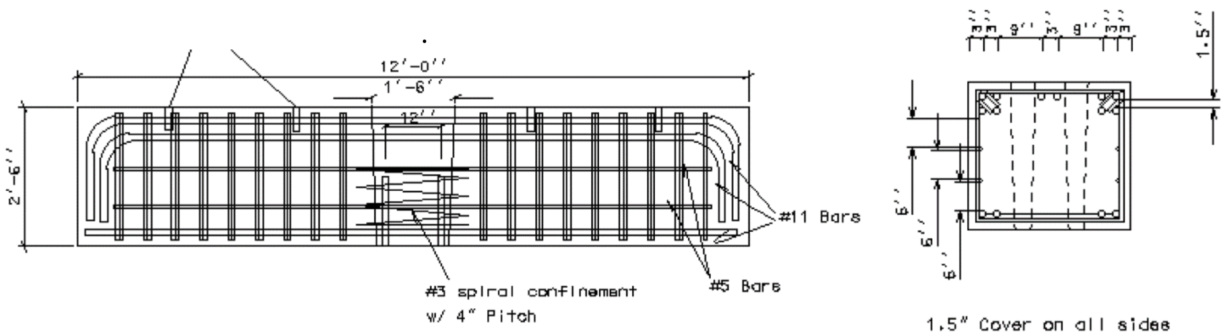


(d) Cap Beam Details for Double-Line Pocket

Figure 1-2. Pocket Specimens for Pullout Test (Matsumoto et al., 2001)

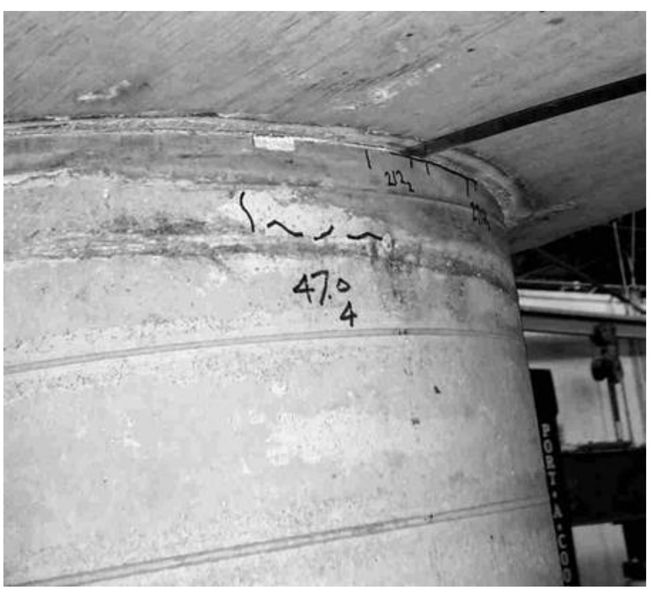


(a) Cap Beam Plan View



(b) Cap Beam Elevation View

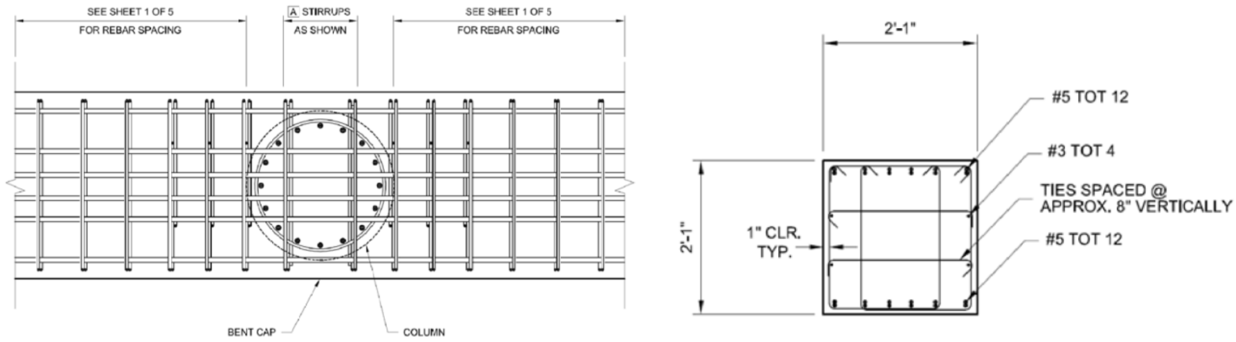
Figure 1-3. Pocket Specimen for Column Test (Matsumoto et al., 2001)



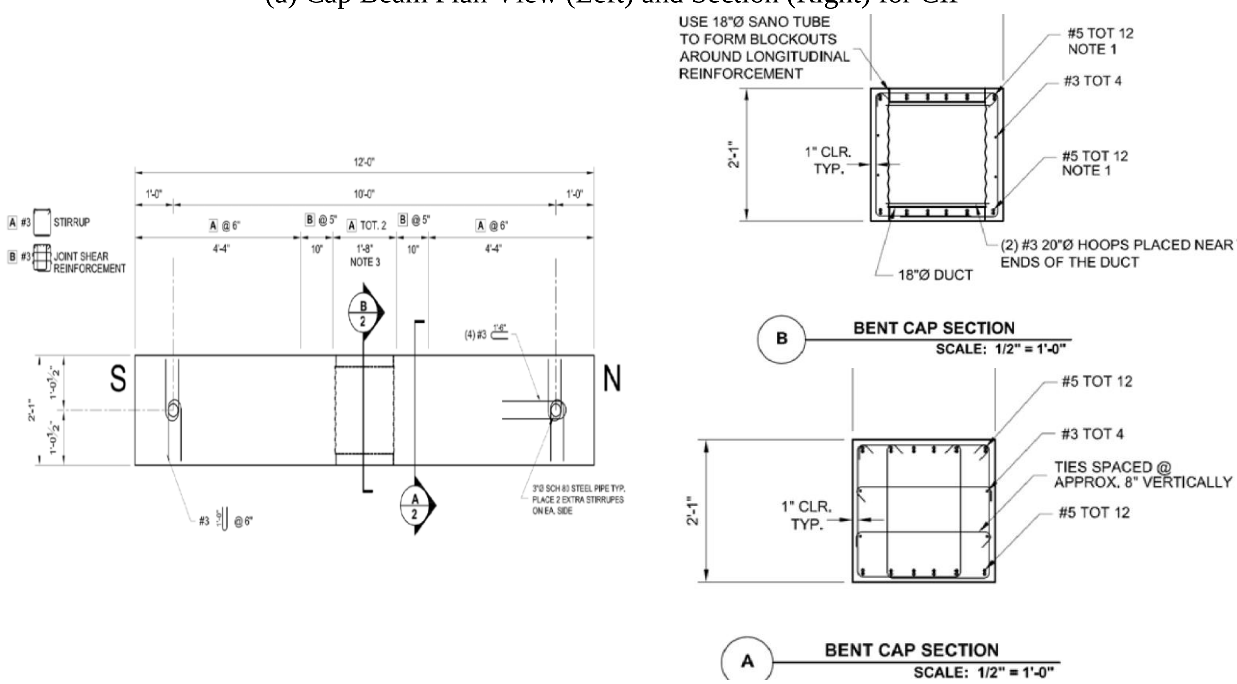
(a) Column-Cap Beam Interface

(b) Column

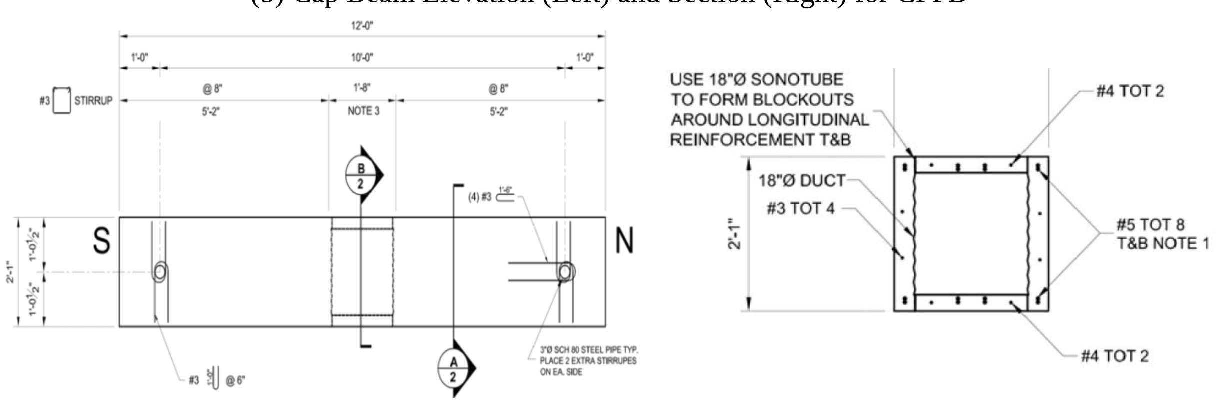
Figure 1-4. Pocket Connections Damage at Failure Load Level (Matsumoto et al., 2001)



(a) Cap Beam Plan View (Left) and Section (Right) for CIP

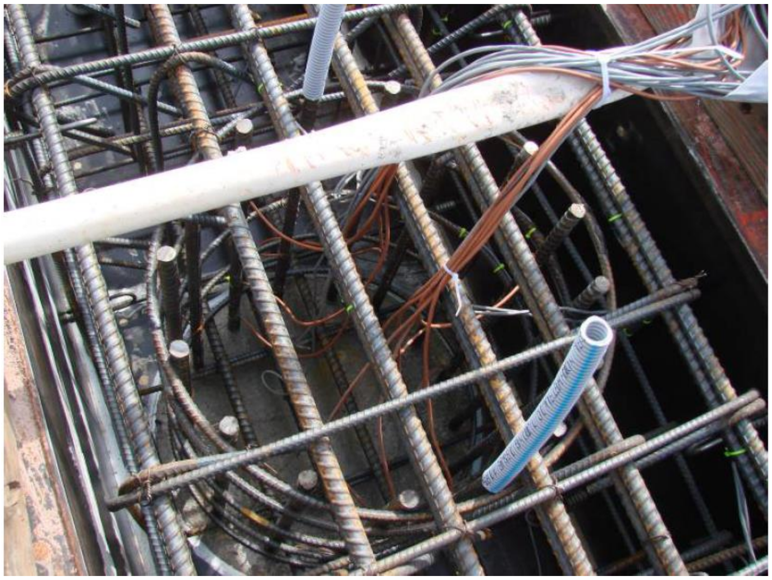


(b) Cap Beam Elevation (Left) and Section (Right) for CPFD



(c) Cap Beam Elevation (Left) and Section B (Right) for CPLD

Figure 1-5. Pocket Connection Details (Restrepo et al., 2011)

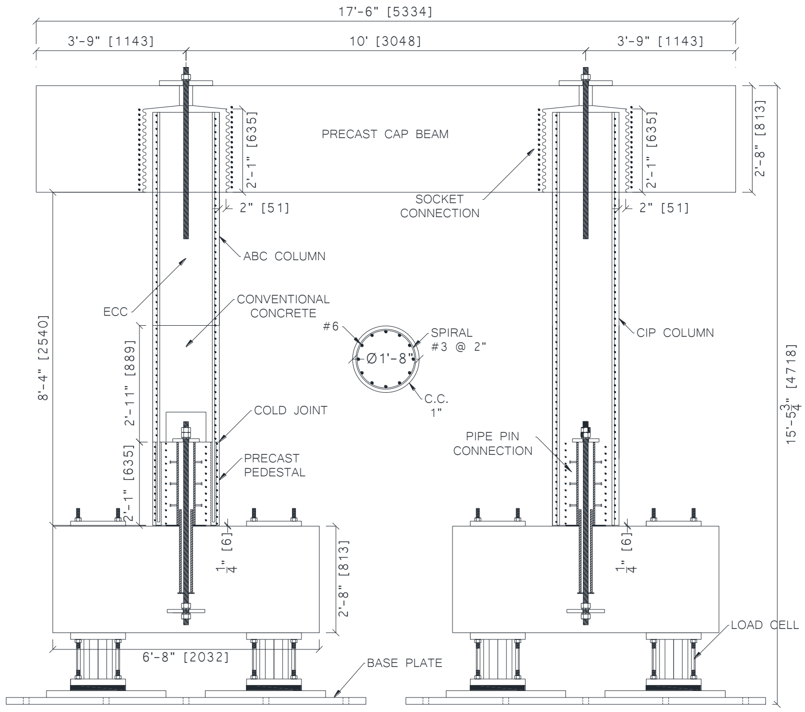


(a) Column to Cap Beam Connection for CIP

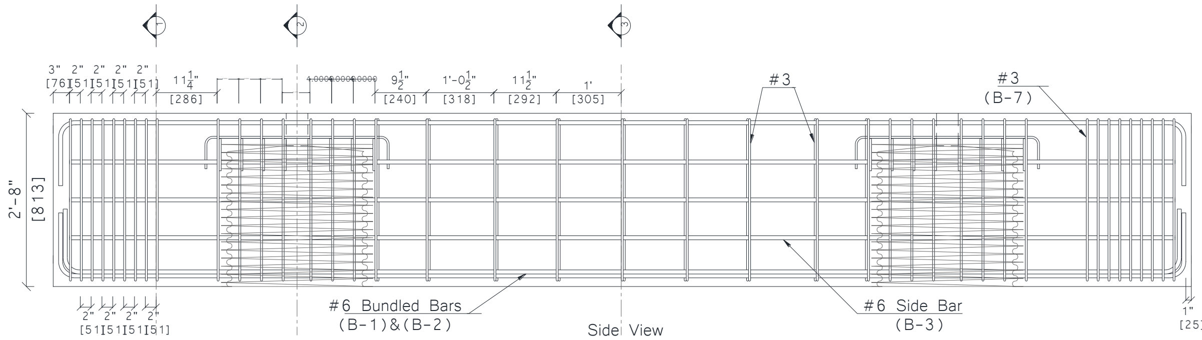
(b) Cap Beam Bar Cage (Left) and Pocket Inside View (Right) for CPFD

(c) Cap Beam Bar Cage (Left) and Pocket Inside View (Right) for CPLD

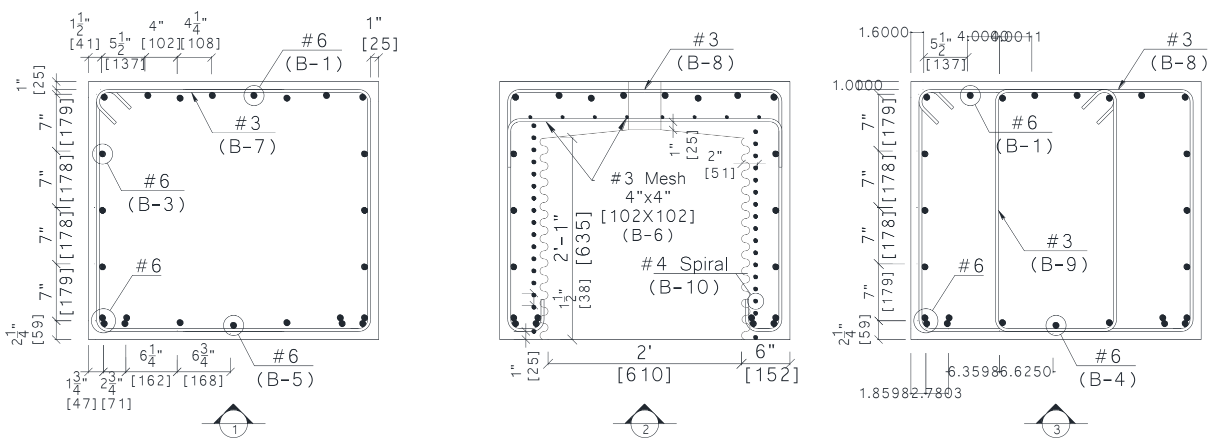
Figure 1-6. Cap Beam Pocket Connections (Restrepo et al., 2011)



(a) Two-Column Bent with Precast Cap Beam



(b) Cap Beam Elevation



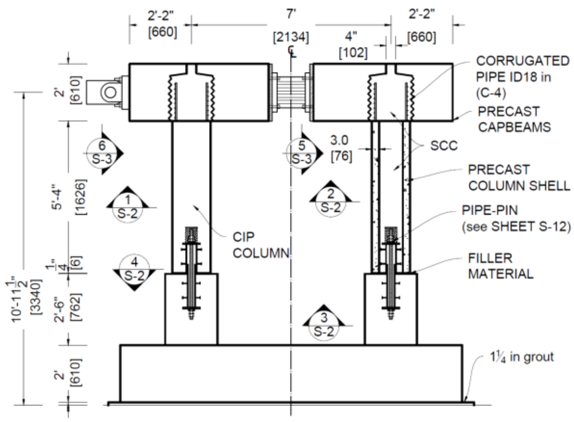
(c) Cap Beam Sections

Figure 1-7. Pocket Connection Details (Mehrsoroush and Saiidi, 2014)

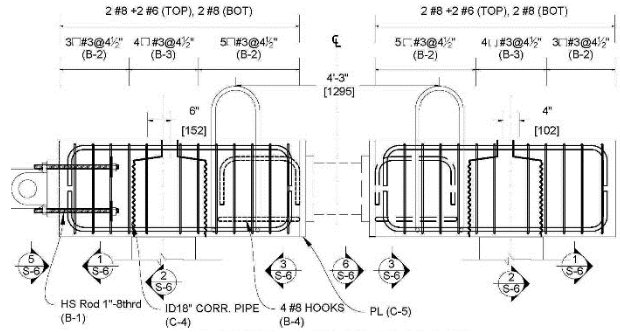
(a) Cap Beam Bar Cage

(b) Corrugated Pocket in Cap Beam

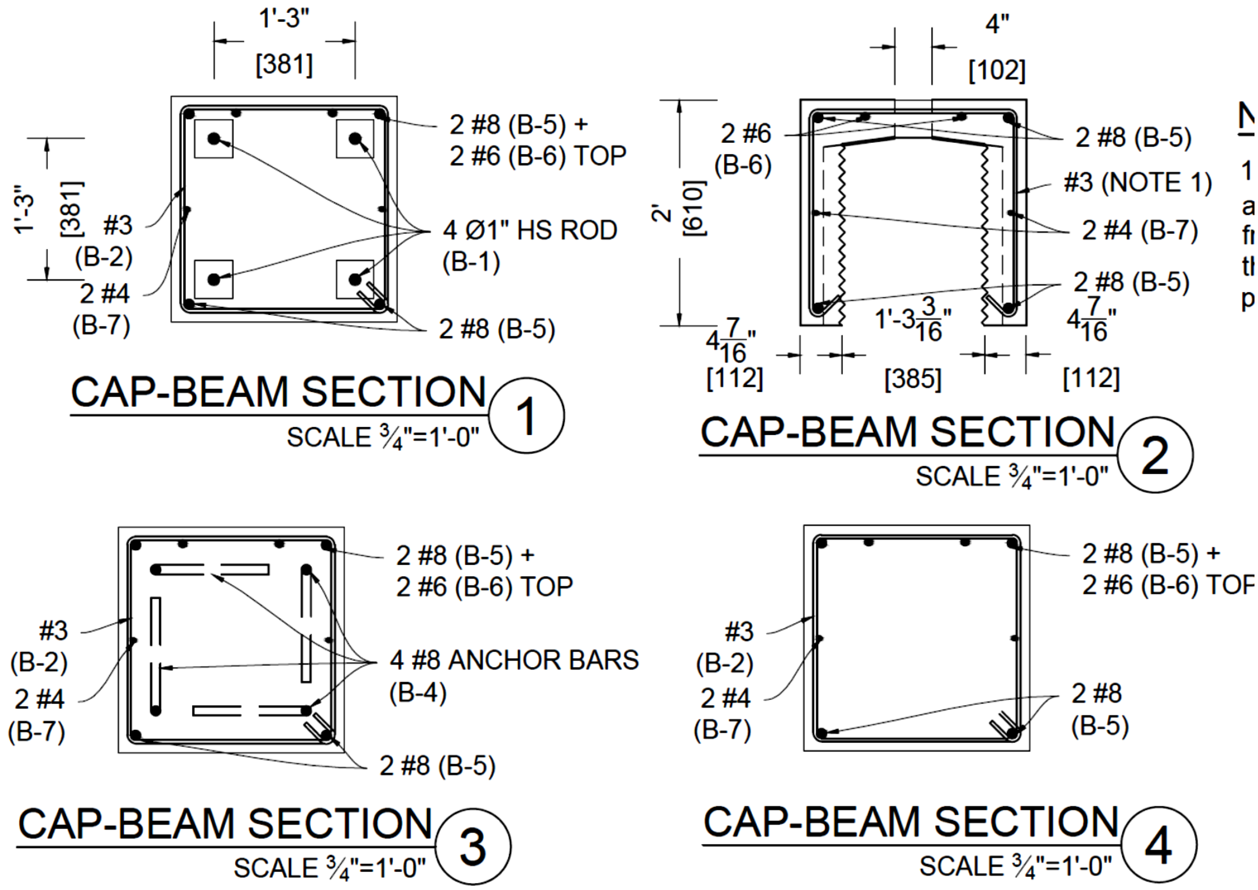
Figure 1-8. Cap Beam Pocket Connections (Mehrsoroush and Saiidi, 2014)



(a) Two-Column Bent with Precast Cap Beams



(b) Cap Beam Elevation



(c) Cap Beam Sections

Figure 1-9. Pocket Connection Details (Mehraein and Saiidi, 2014)

Figure 1-10. Cap Beam Pocket Connections (Mehraein and Saiidi, 2014)

(a) Column Embedded in Footing Pocket

(b) Final Bent

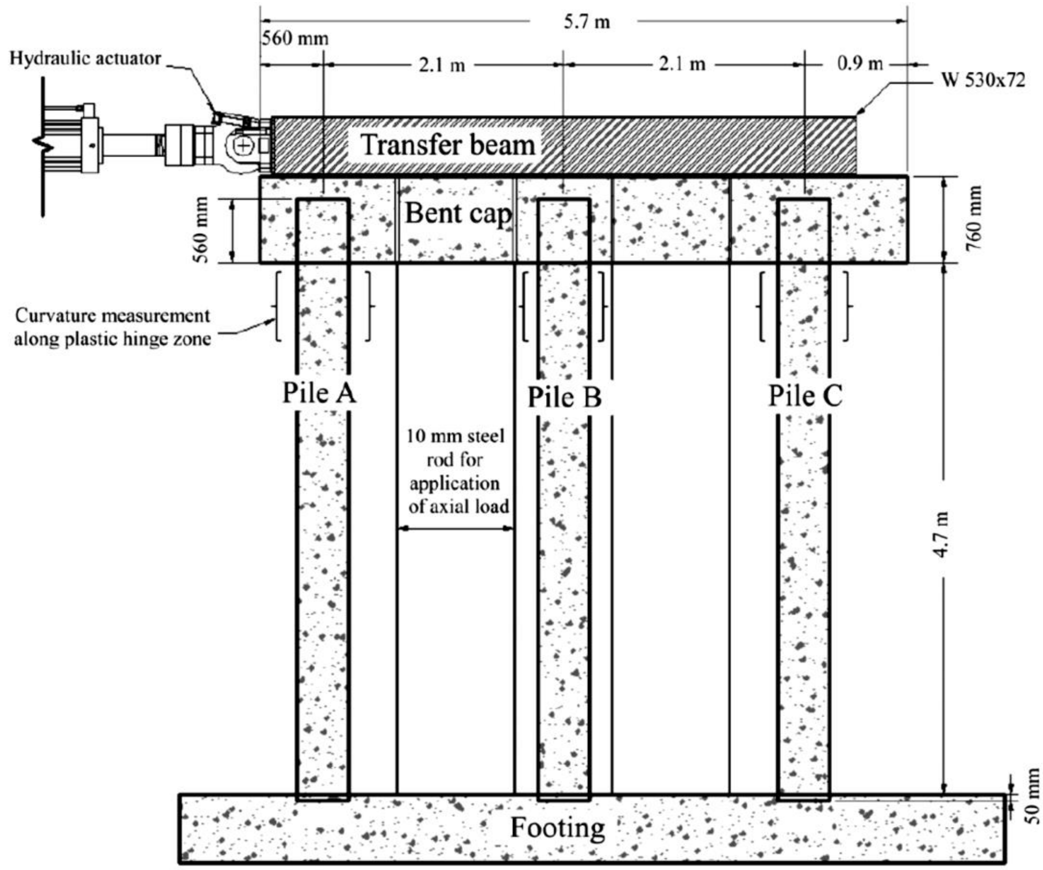
Figure 1-11. Footing Pocket Connections (Motaref et al., 2011)

(a) Column Embedded in Footing (b) Final Bridge

Figure 1-12. Pocket Connections with Cast-in-Place Footings (Khaleghi et al., 2012)

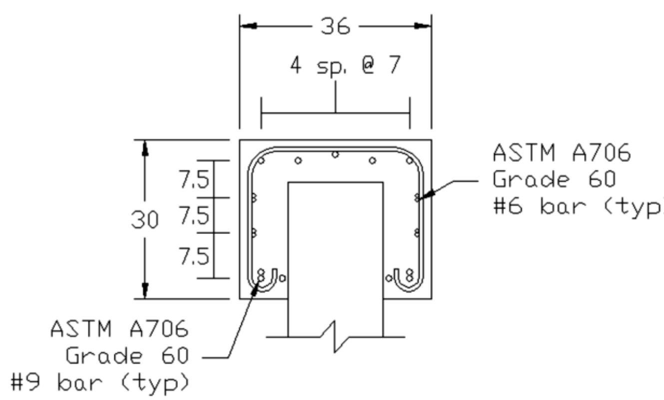
(a) Column Embedded in Footing (b) Final Bridge

Figure 1-13. Footing Pocket Connections (Kavianipour and Saiidi, 2013)



(a) Pile to Cap Specimen (Larosche et al., 2014b)

(b) Pile Embedded into Bent Cap (Cukrov and Sanders, 2012)

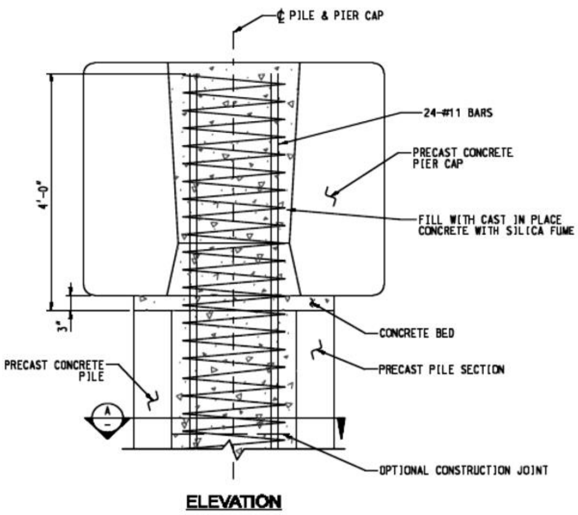


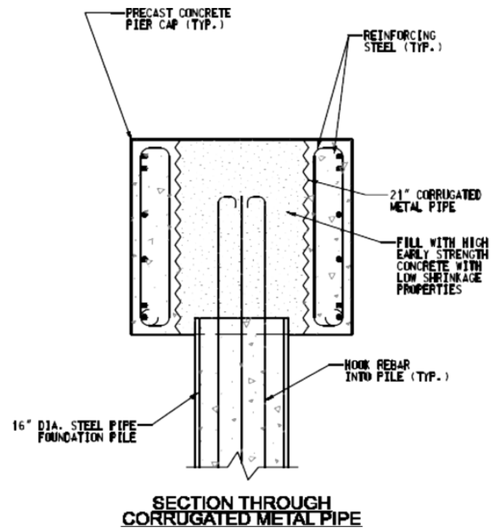
(c) Bent Cap Section at the center-line of Pile C (Cukrov and Sanders, 2012)

Figure 1-14. Pile-to-Cap Pocket Connection

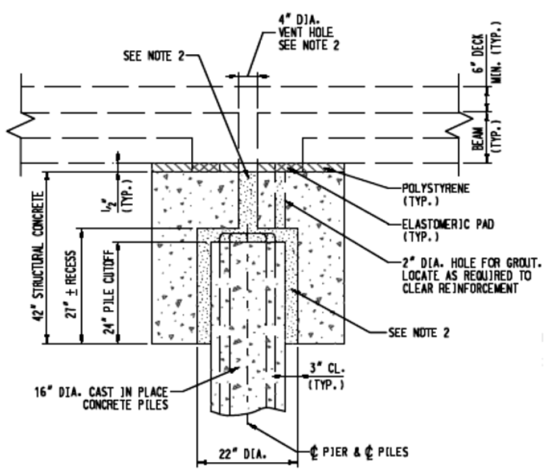
(a) Redfish Bay Project (b) US 290 Ramp E-3 Project

Figure 1-15. Field Application of Cap Beam Pocket Connections in Texas (Brenes et al., 2006)





(a) Florida DOT (b) Iowa DOT



(b) Minnesota DOT

Figure 1-16. Field Application of Cap Beam Pocket Connections (Marsh et al., 2011)

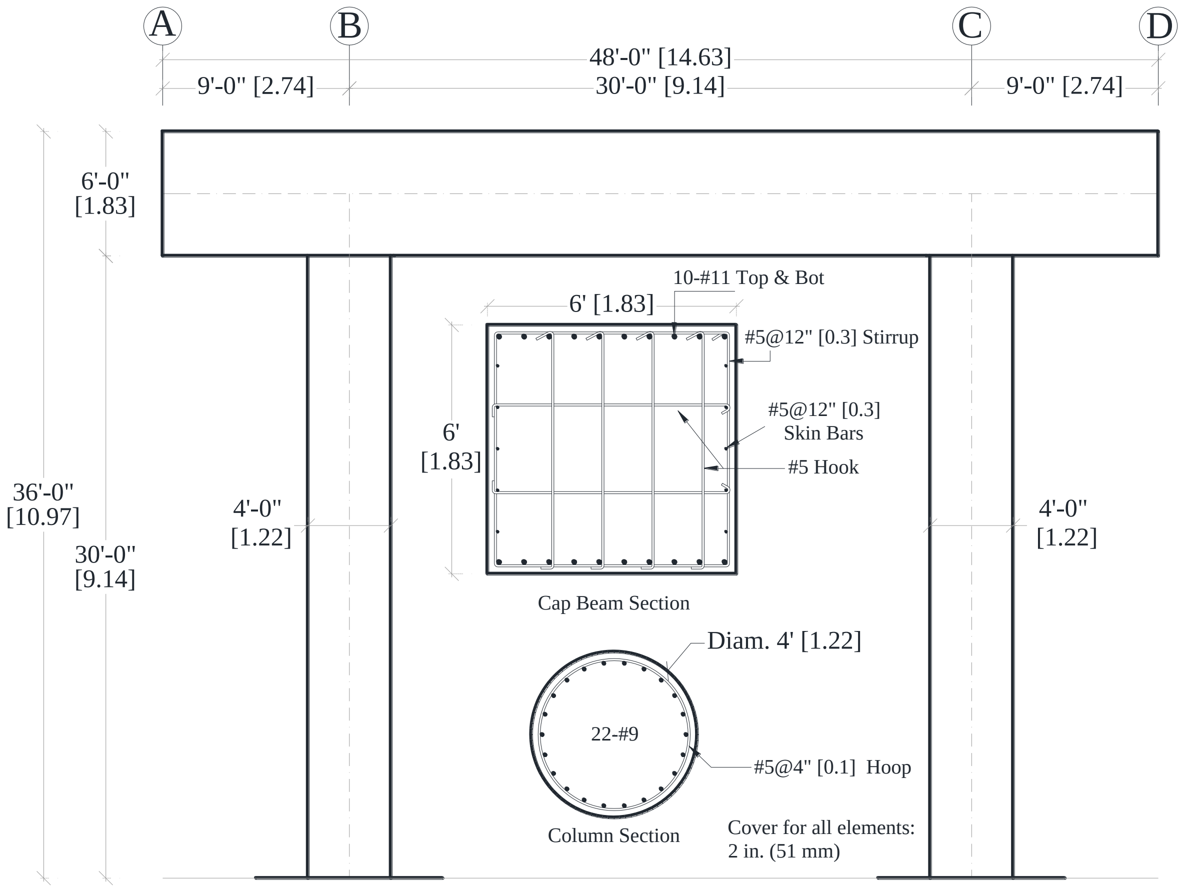


Figure 2-1. Reference Two-Column Bent Details, units: *ft* [*m*]

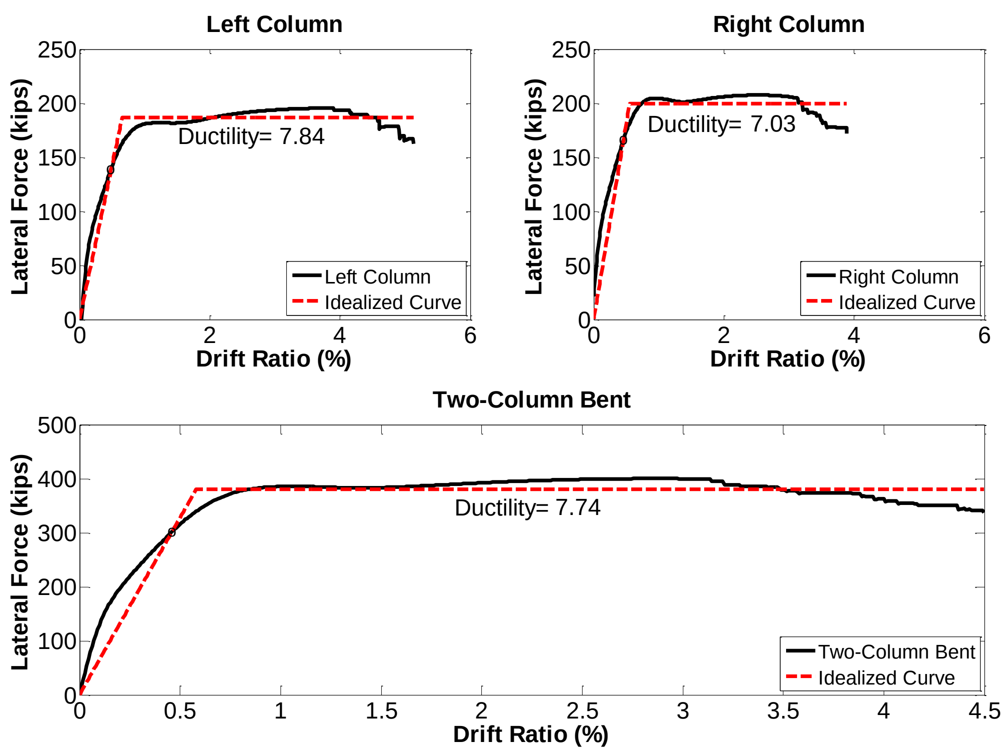


Figure 2-2. Pushover Response of Reference Bent for Loading from Left

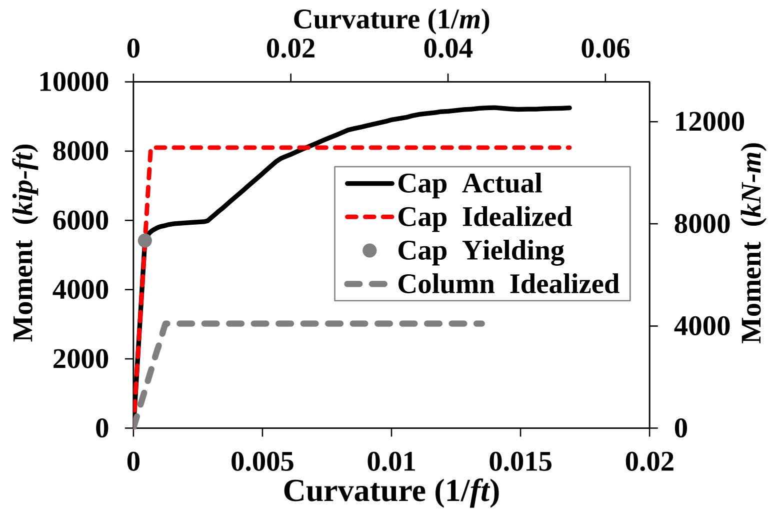


Figure 2-3. Moment-Curvature Relationships for Reference Bent Cap Beam and Columns

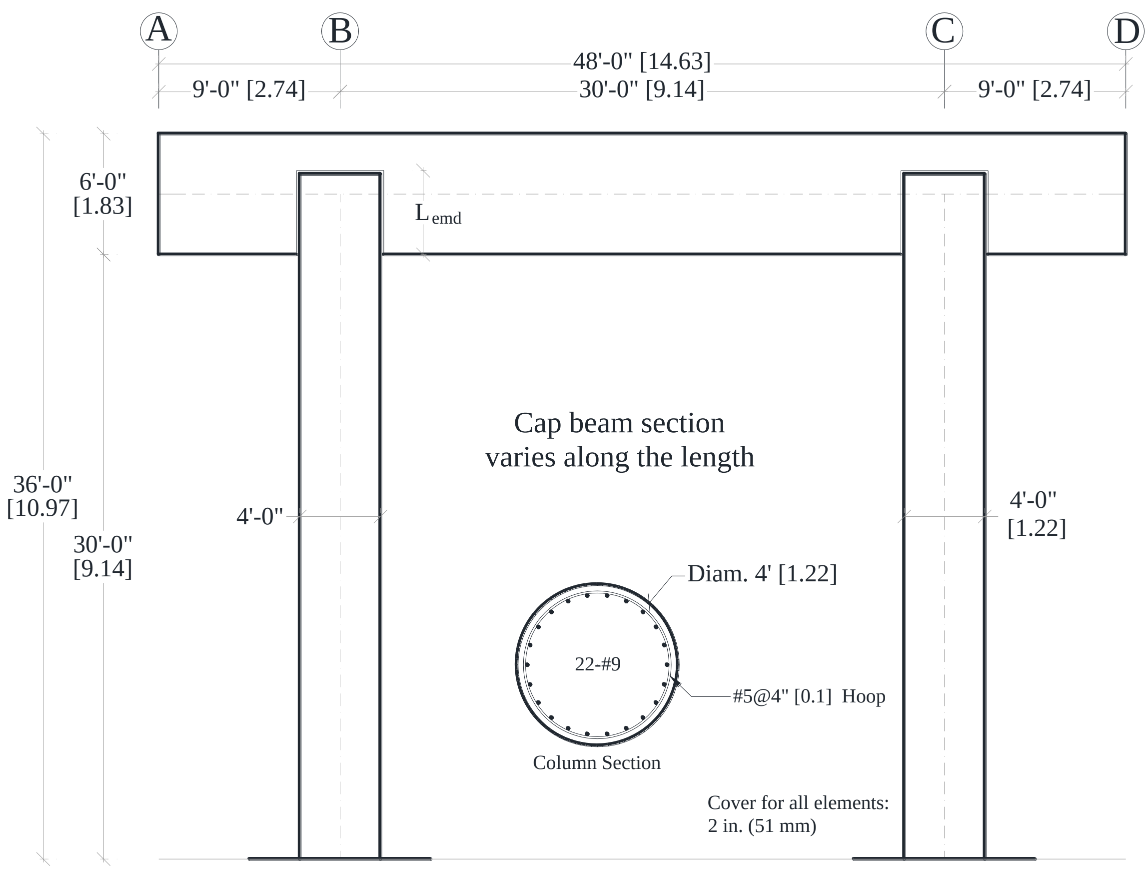
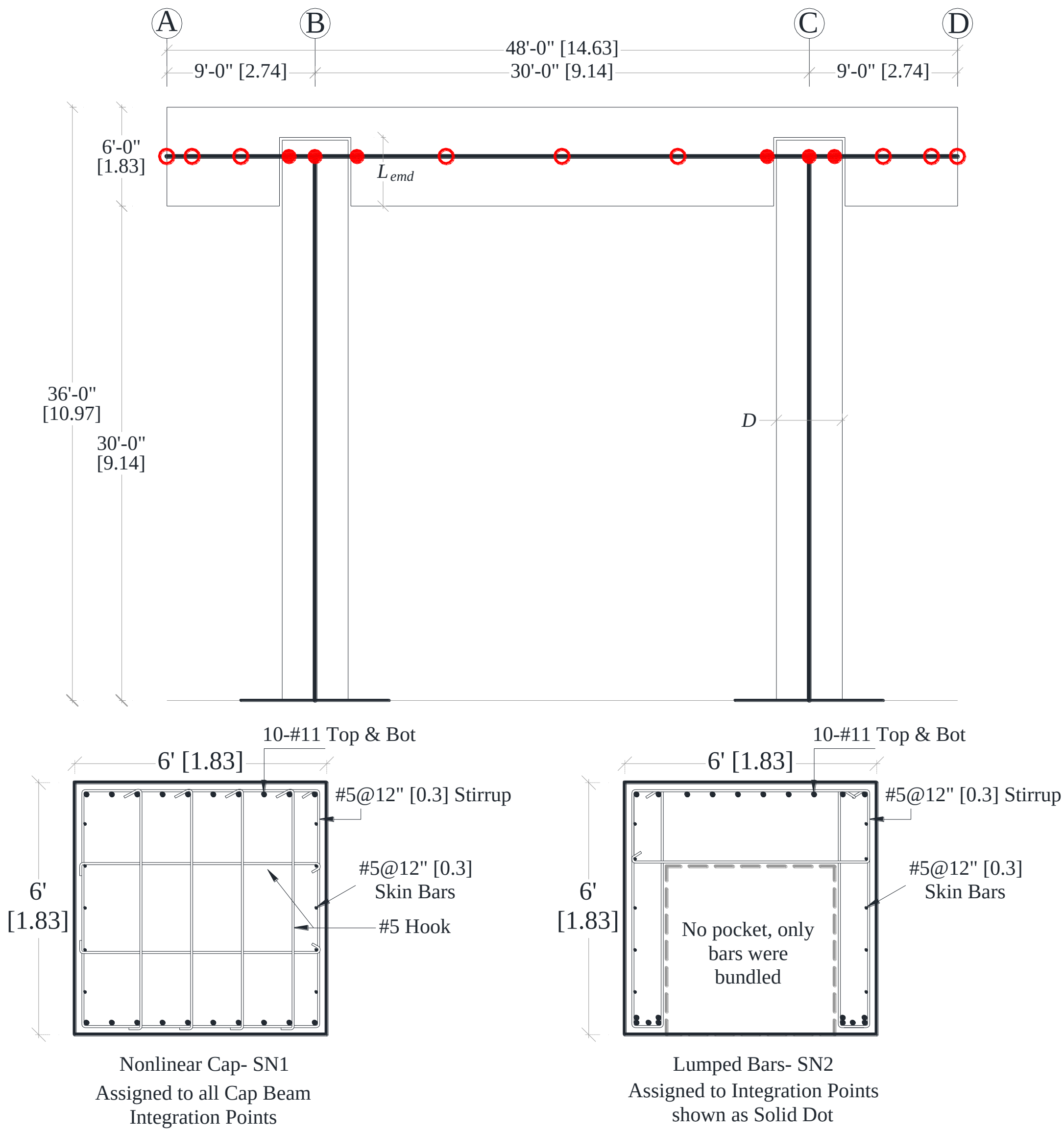
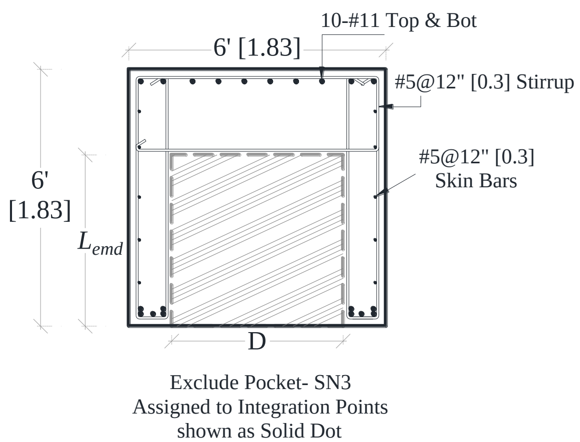


Figure 2-4. Reference Two-Column Bent Details with Pocket Connections, units: *ft* [*m*]





Table

Scenario No.	L_{emd}
3	1.*D*
4	1.1*D*
5	1.2*D*
6	1.3*D*
7	1.4*D*
8	1.5*D*

Figure 2-5. Different Scenarios for Bent with Pocket Connections, units: *ft* [*m*]

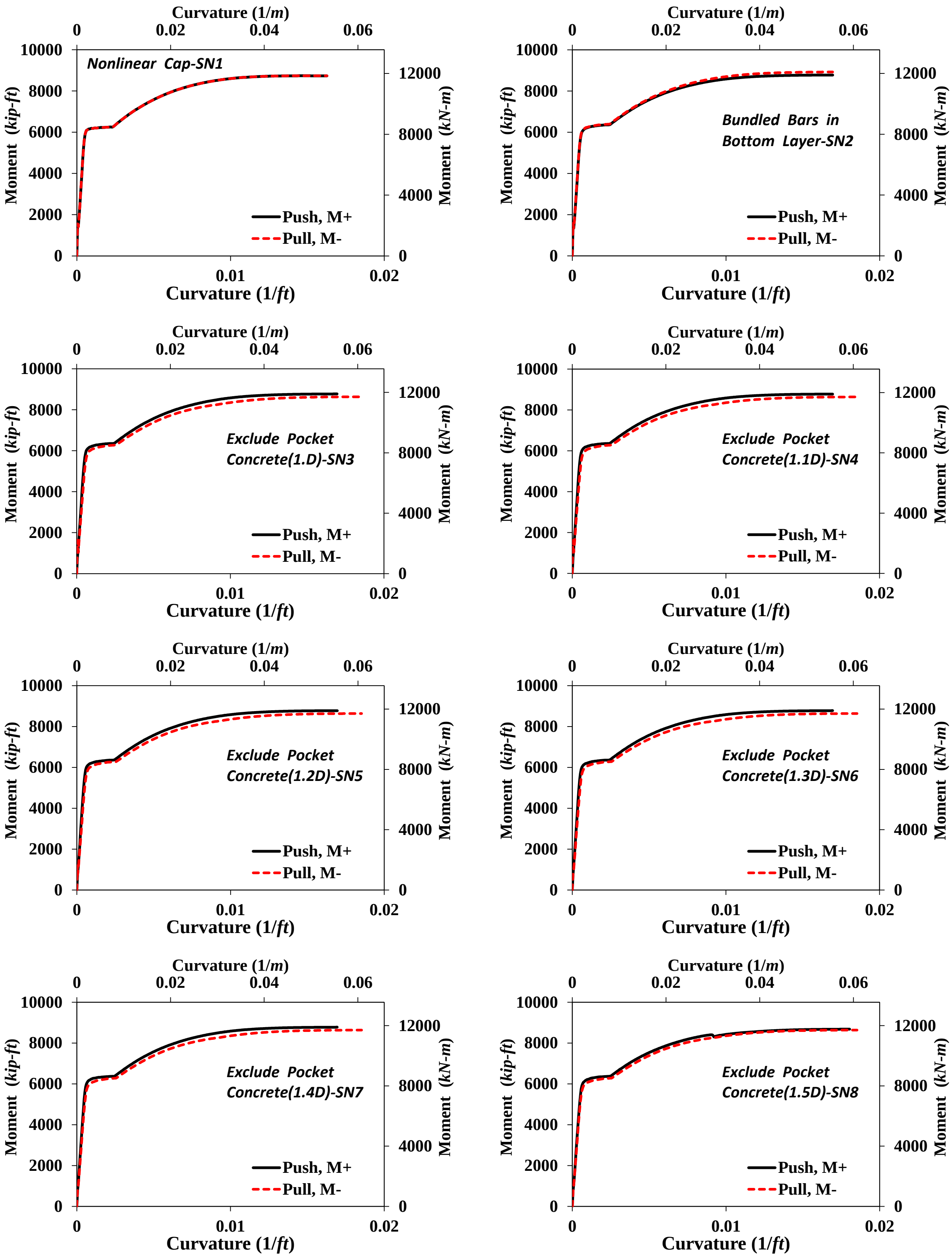


Figure 2-6. Moment-Curvature Relationships for Reference Cap Beam

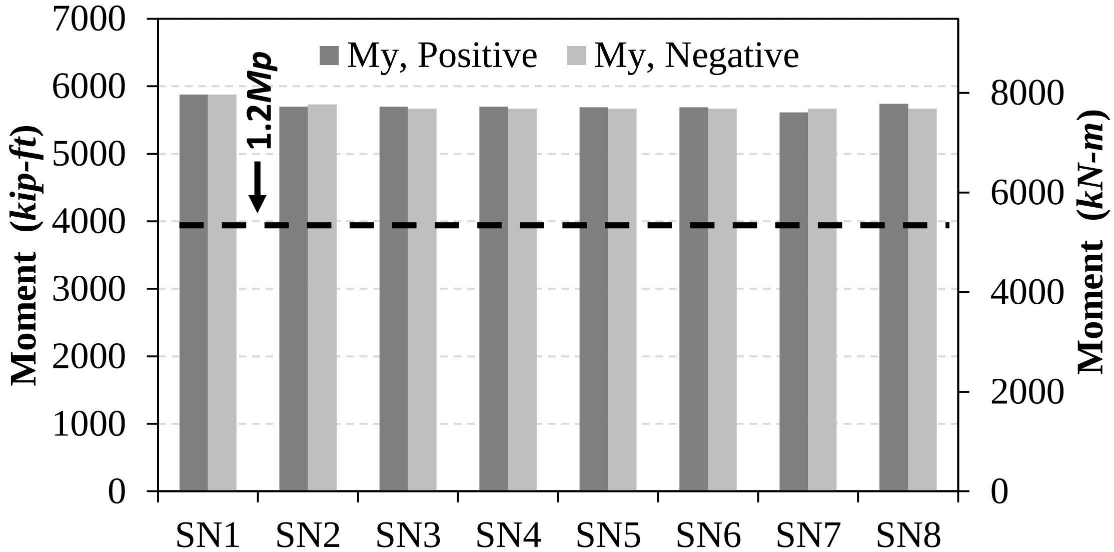


Figure 2-7. First Yield Moment for Reference Cap Beam for Different Scenarios

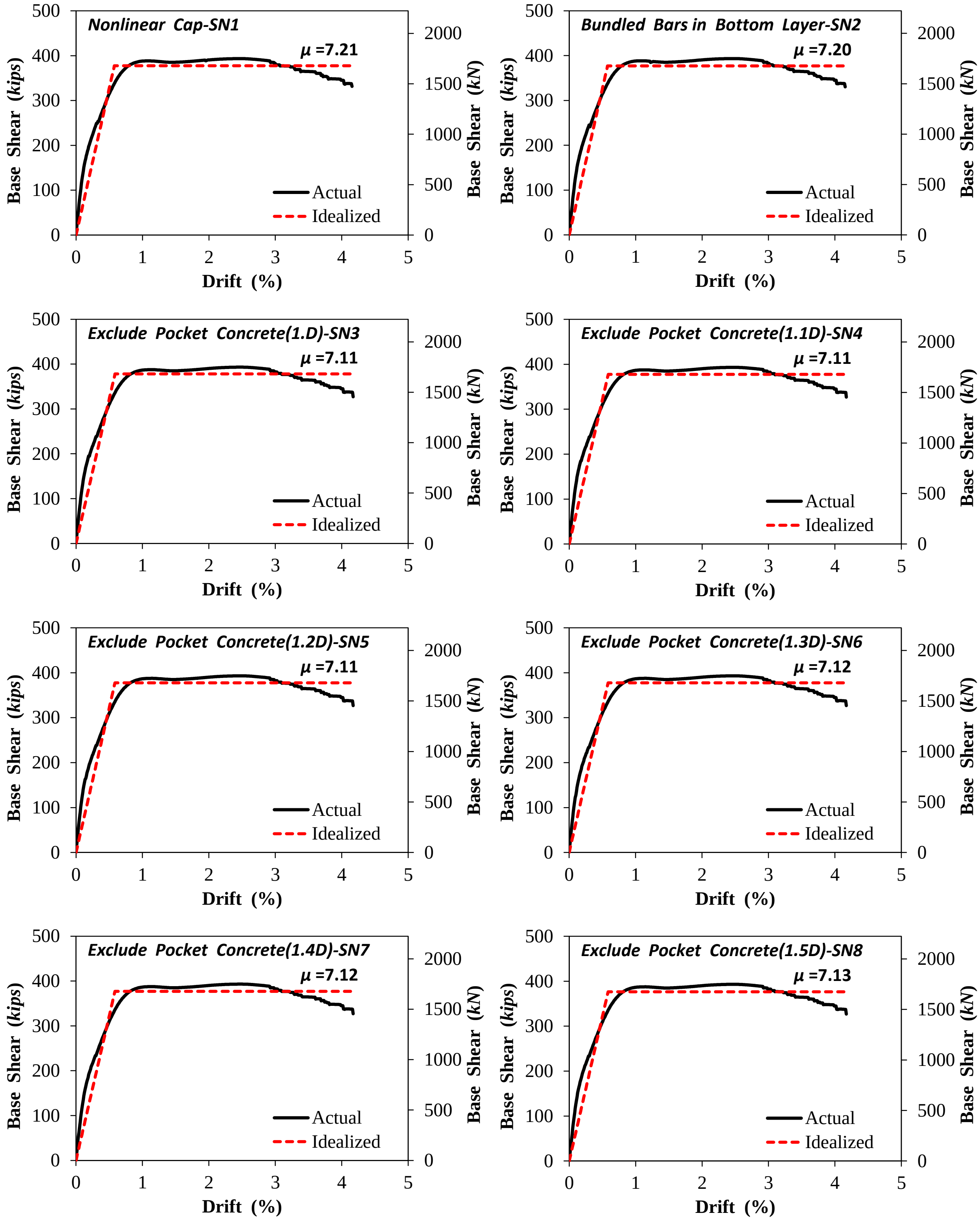


Figure 2-8. Pushover Curves for Reference Two-Column Bent

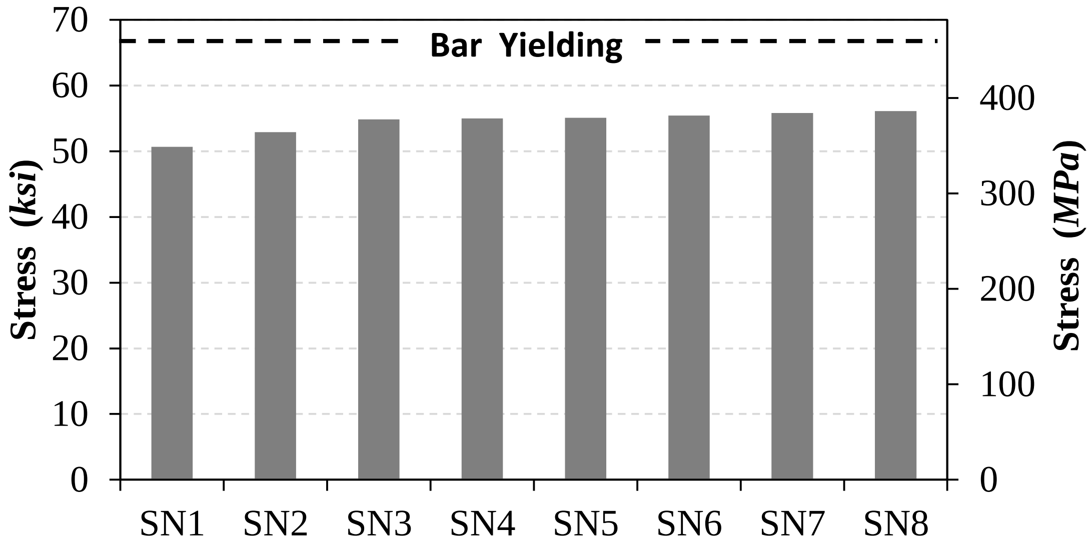


Figure 2-9. Peak Tensile Strains of Cap Beam Steel Bars for Different Scenarios

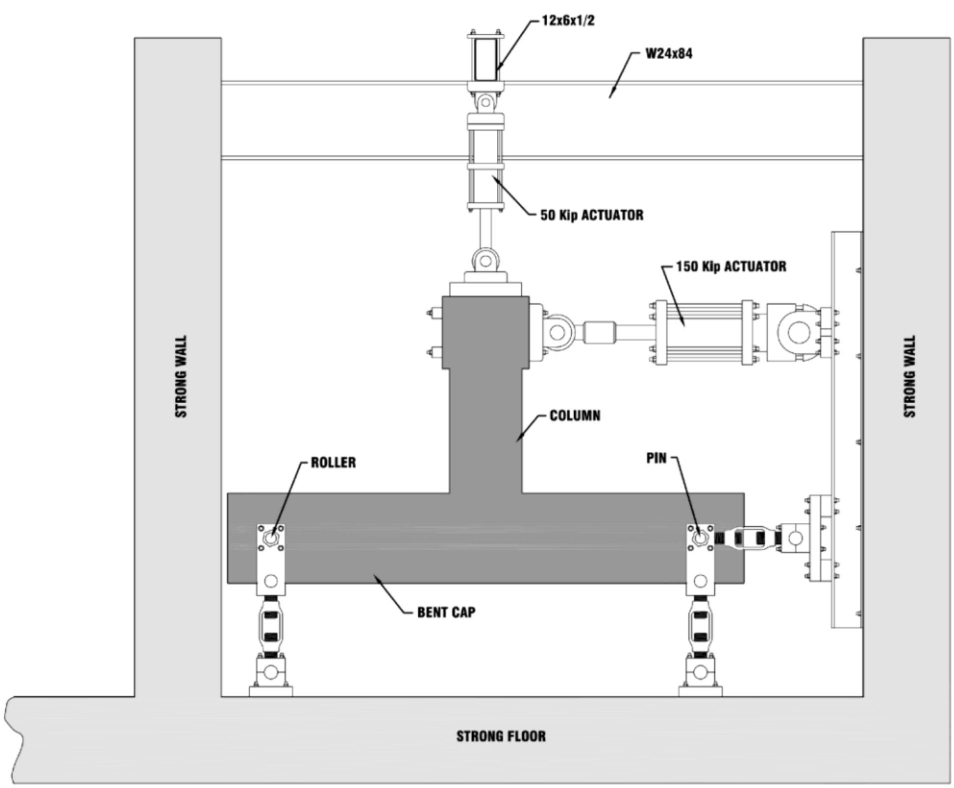
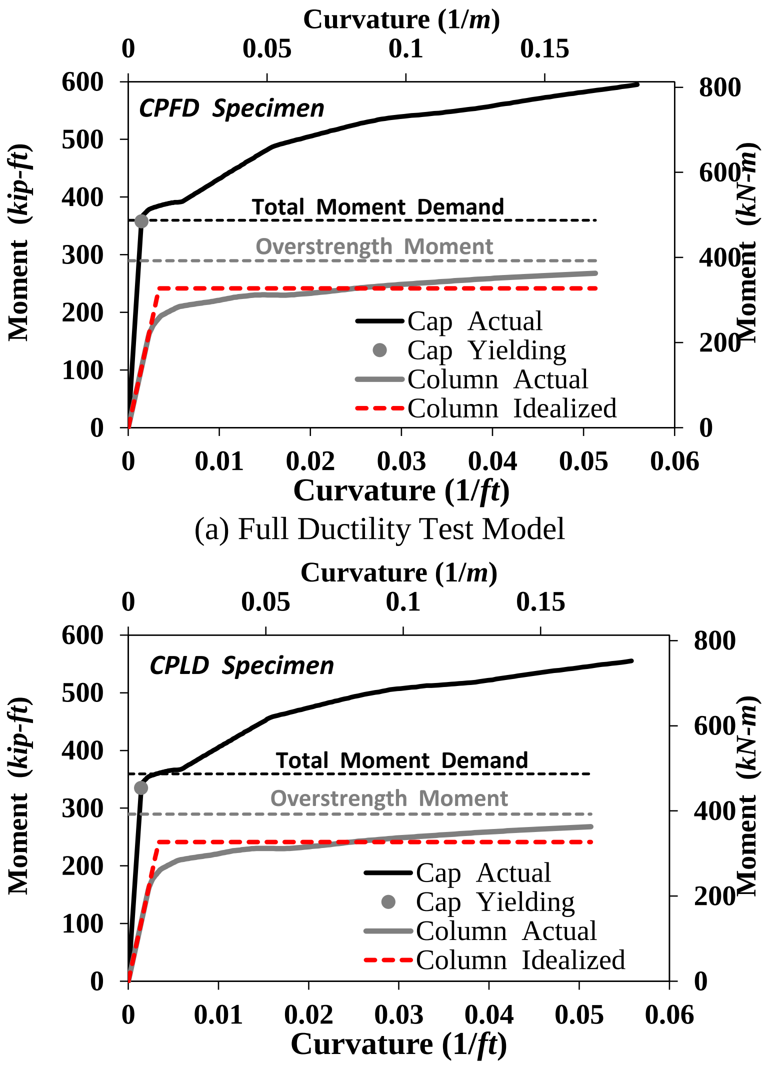


Figure 2-10. Cap Beam Pocket Connection Test Setup (Restrepo et al., 2011)



(a) Full Ductility Test Model

(b) Limited Ductility Test Mode

Figure 2-11. Moment-Curvature Relationships for Cap Pocket Test Models in Restrepo et al. (2011)

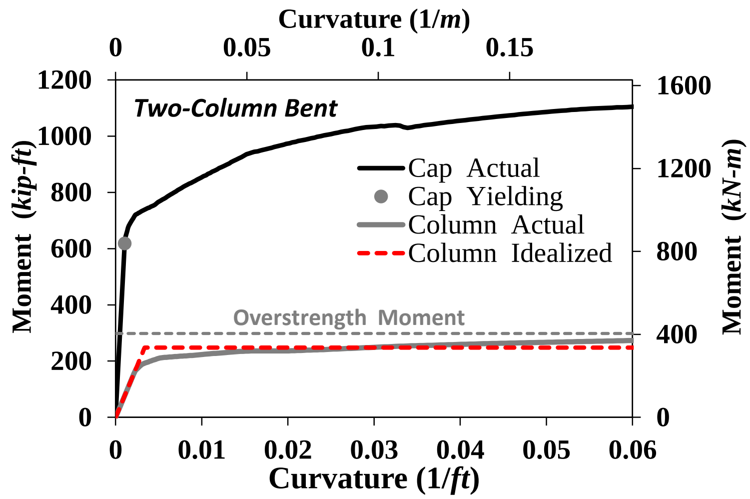


Figure 2-12. Moment-Curvature Relationships for Bent Tested by Mehrsoroush and Saiidi (2014)

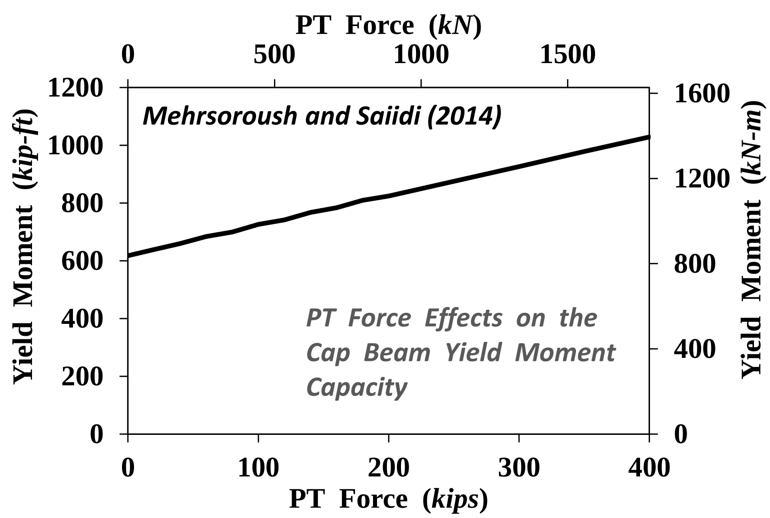


Figure 2-13. Post-tensioning Force Effects on Cap Beam Yield Moment Capacity

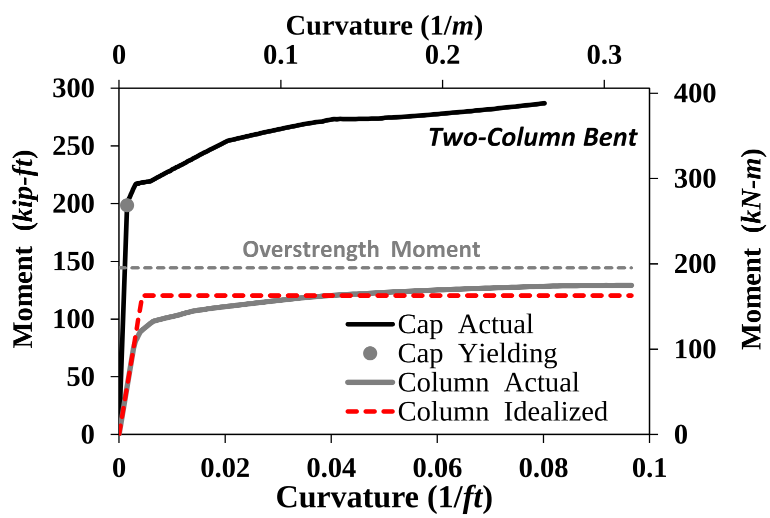


Figure 2-14. Moment-Curvature Relationships for Bent Tested by Mehraein and Saiidi (2014)

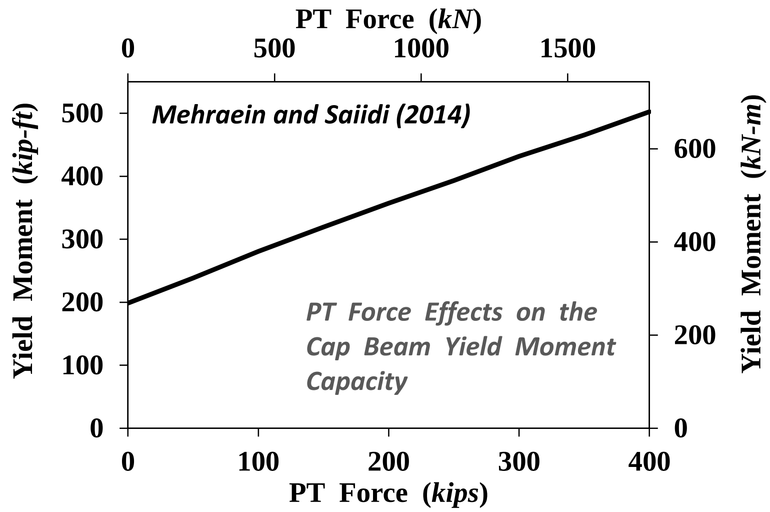
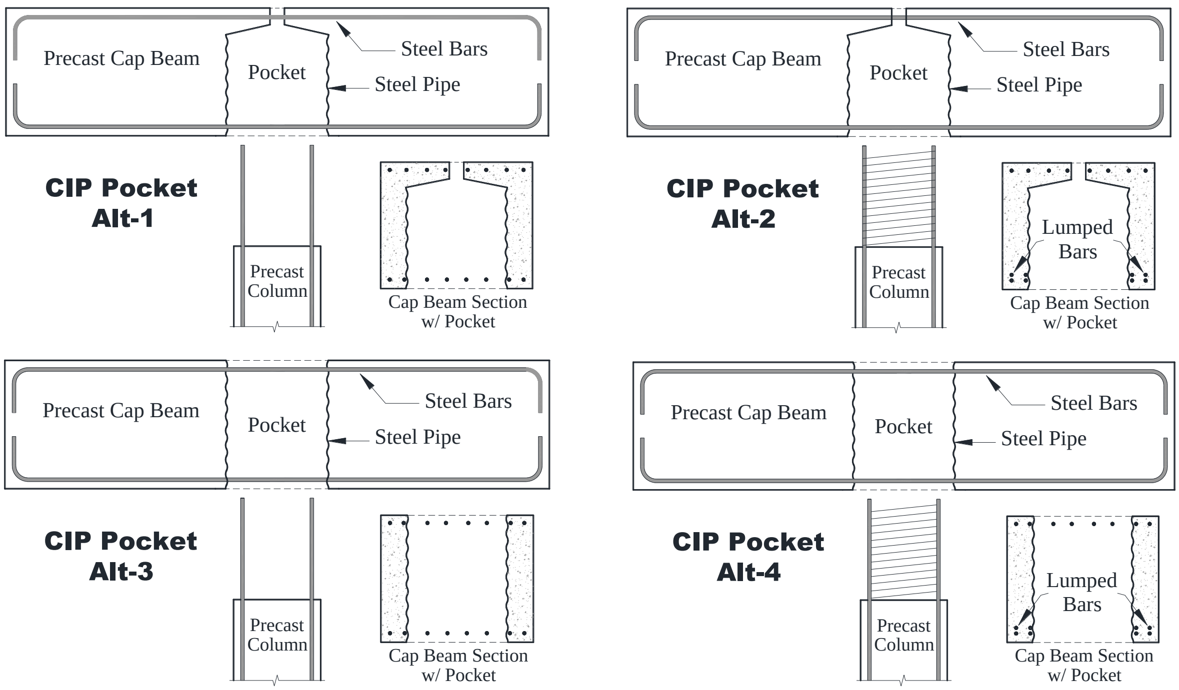
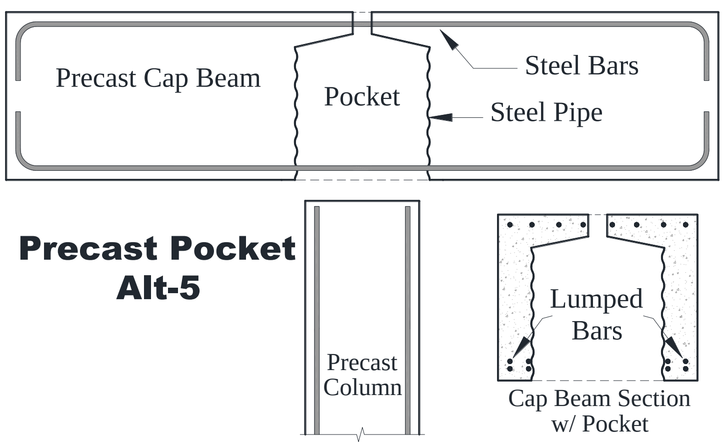


Figure 2-15. Post-tensioning Force Effects on Cap Beam Yield Moment Capacity



(a) Cast-in-Place Pocket Connections



(b) Precast Pocket Connection

Figure 3-1. Different Detailing for Pocket Connections

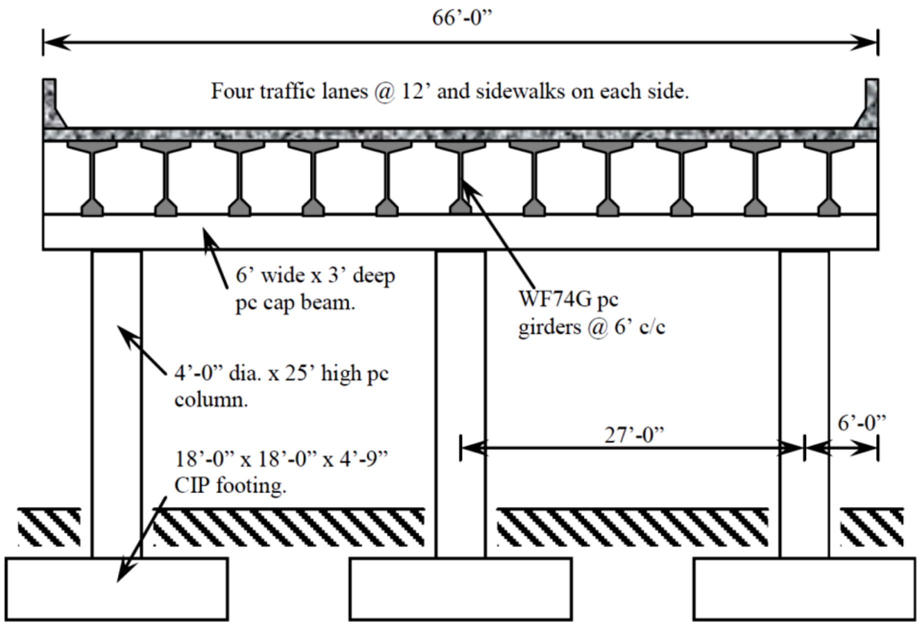


Figure 3-2. Reference Cast-in-Place Bent (Marsh et al. 2011)

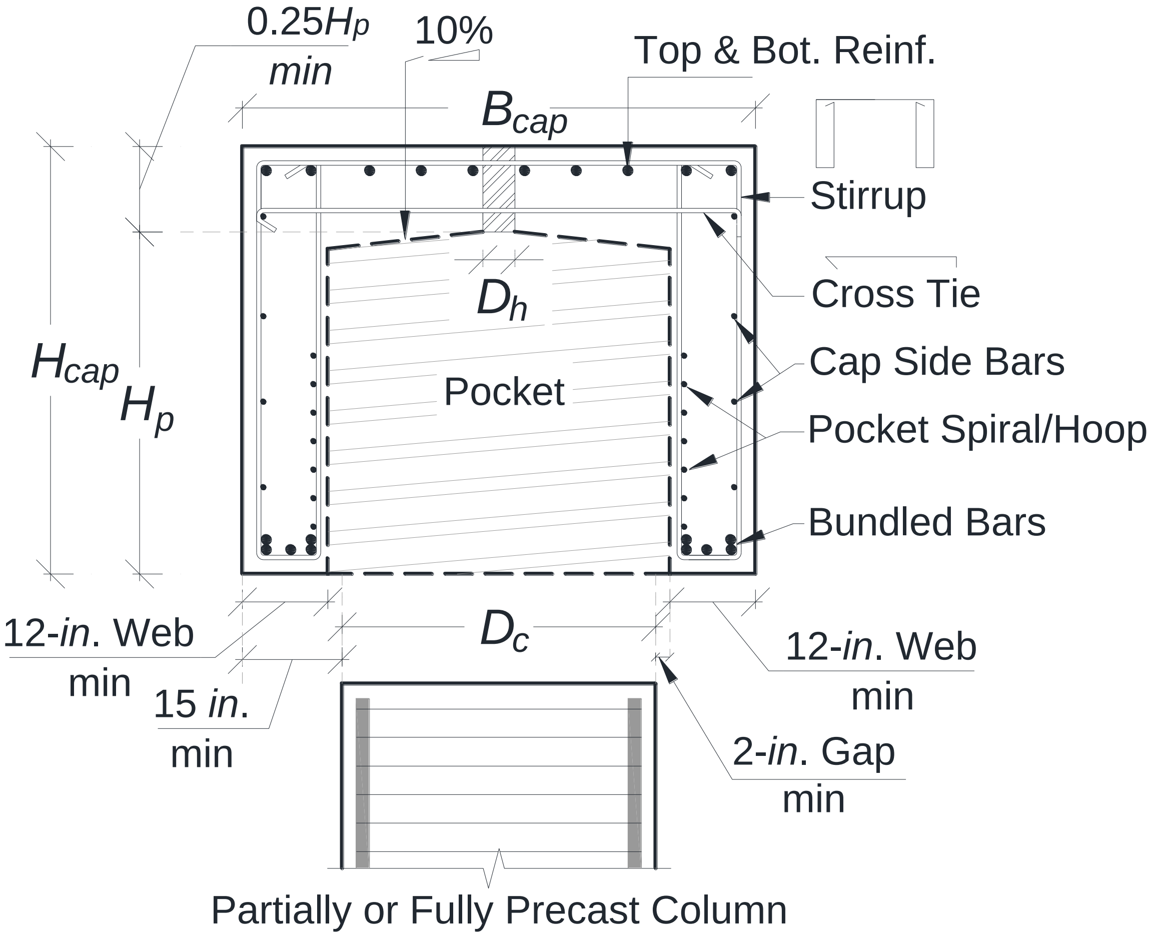
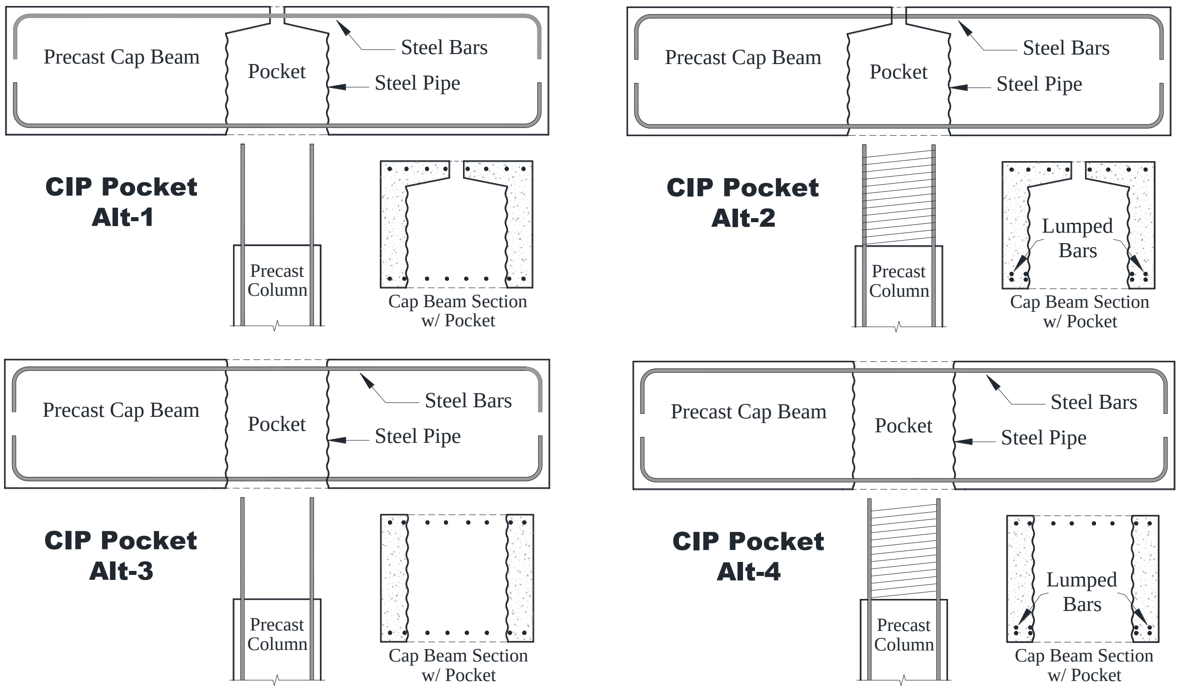
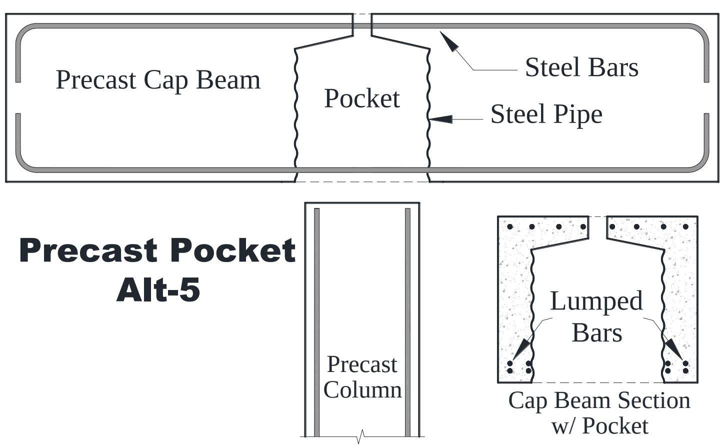


Figure R-1. Proposed Dimension for Cap Beams with Pocket

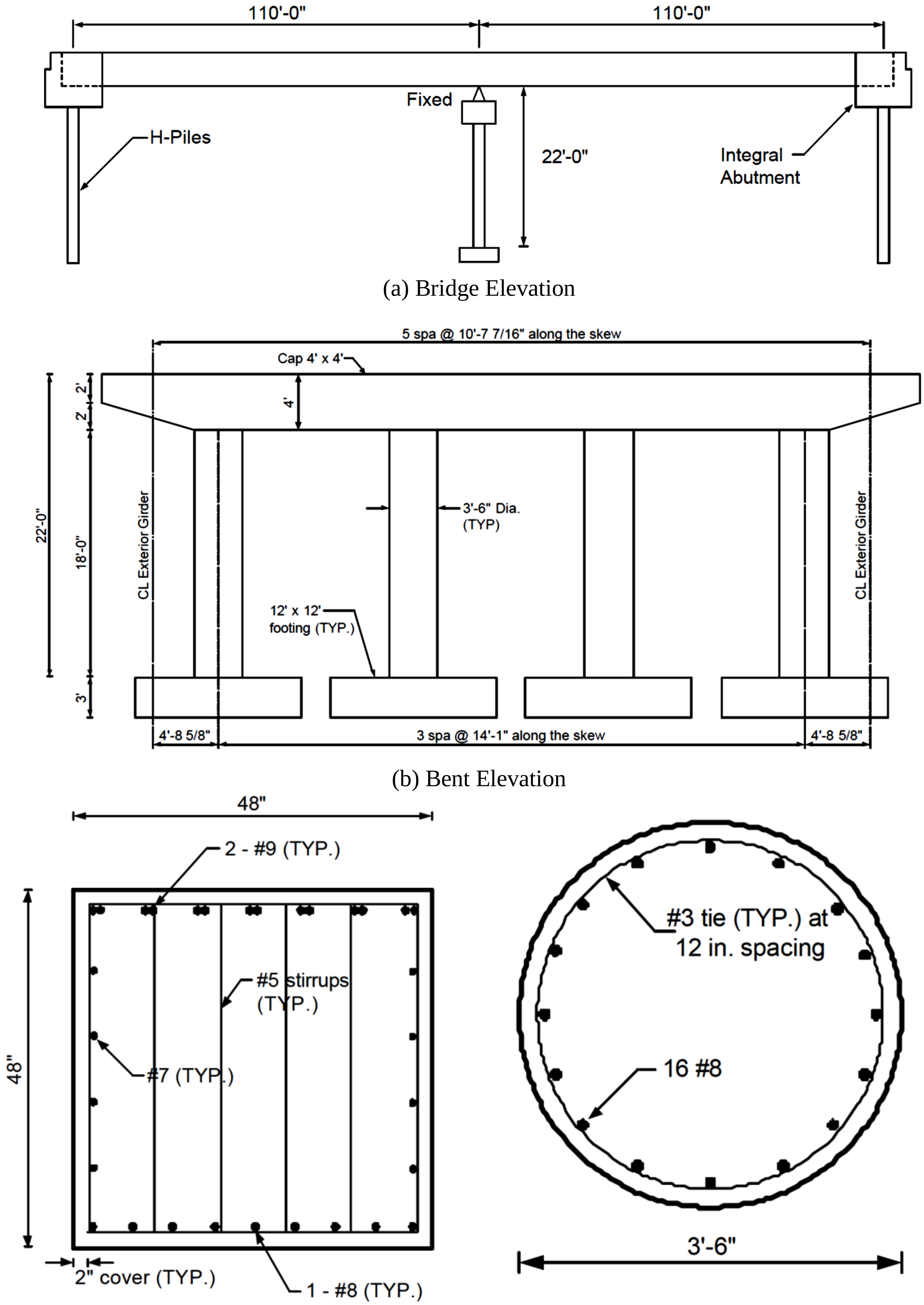


(a) Cast-in-Place Pocket Connections



(b) Precast Pocket Connection

Figure C-1. Proposed Detailing for Pocket Connections



(a) Bridge Elevation

(b) Bent Elevation

(c) Cap Beam Section

(d) Column Section

Figure 5-1. Reference Cast-in-Place Bridge

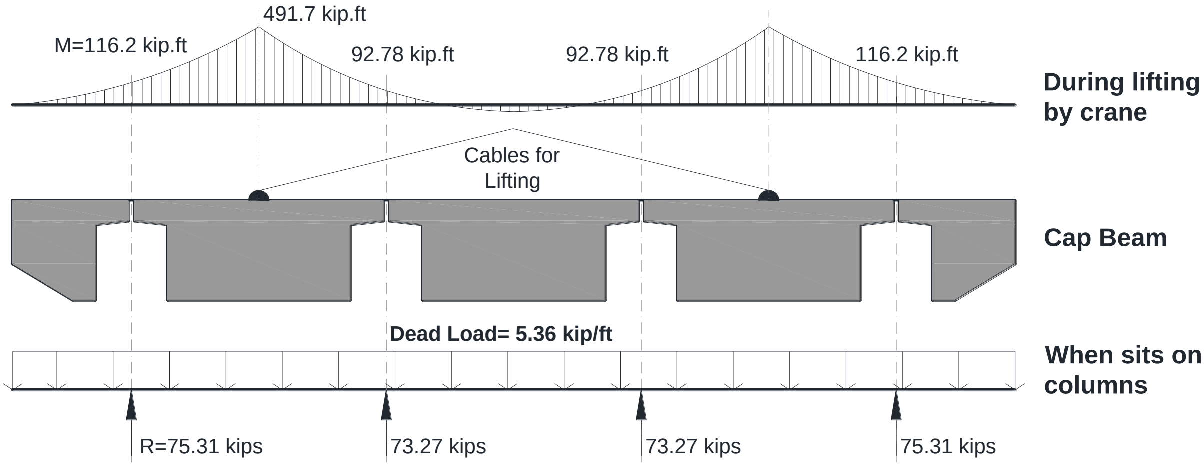


Figure 5-2. Bent Cap Moment and Punching Forces during Lifting and Installing

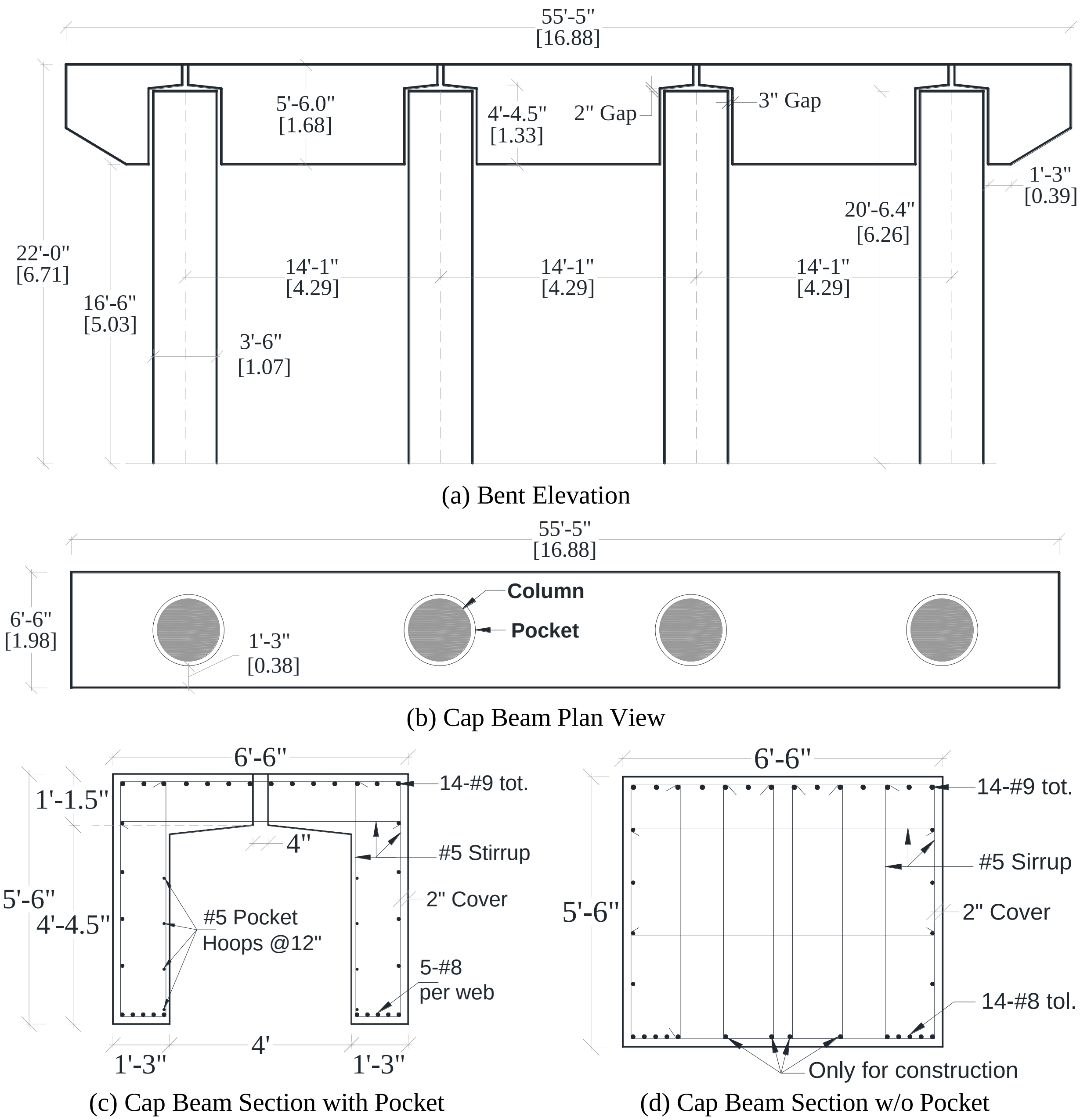


Figure 5-3. Precast Bent Cap

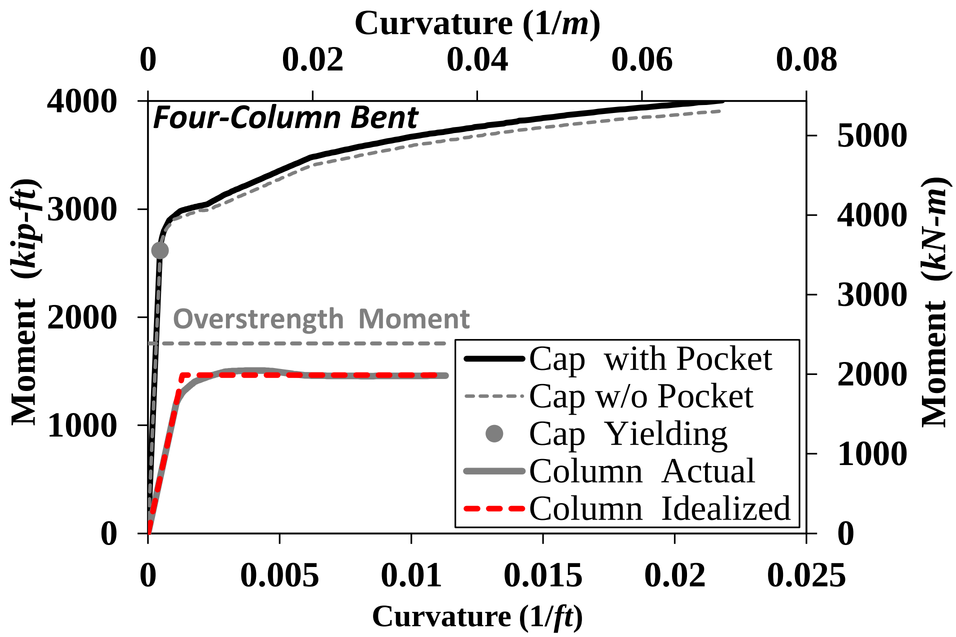


Figure 5-4. Precast Bent Cap Moment-Curvature Relationship

List of CCEER Publications

Report No.	Publication
CCEER-84-1	Saiidi, M., and R. Lawver, “User's Manual for LZAK-C64, A Computer Program to Implement the Q-Model on Commodore 64,” Civil Engineering Department, Report No. CCEER-84-1, University of Nevada, Reno, January 1984.
CCEER-84-1 Reprint	Douglas, B., Norris, G., Saiidi, M., Dodd, L., Richardson, J. and Reid, W., “Simple Bridge Models for Earthquakes and Test Data,” Civil Engineering Department, Report No. CCEER-84-1 Reprint, University of Nevada, Reno, January 1984.
CCEER-84-2	Douglas, B. and T. Iwasaki, “Proceedings of the First USA-Japan Bridge Engineering Workshop,” held at the Public Works Research Institute, Tsukuba, Japan, Civil Engineering Department, Report No. CCEER-84-2, University of Nevada, Reno, April 1984.
CCEER-84-3	Saiidi, M., J. Hart, and B. Douglas, “Inelastic Static and Dynamic Analysis of Short R/C Bridges Subjected to Lateral Loads,” Civil Engineering Department, Report No. CCEER-84-3, University of Nevada, Reno, July 1984.
CCEER-84-4	Douglas, B., “A Proposed Plan for a National Bridge Engineering Laboratory,” Civil Engineering Department, Report No. CCEER-84-4, University of Nevada, Reno, December 1984.
CCEER-85-1	Norris, G. and P. Abdollaholiaee, “Laterally Loaded Pile Response: Studies with the Strain Wedge Model,” Civil Engineering Department, Report No. CCEER-85-1, University of Nevada, Reno, April 1985.
CCEER-86-1	Ghusn, G. and M. Saiidi, “A Simple Hysteretic Element for Biaxial Bending of R/C in NEABS-86,” Civil Engineering Department, Report No. CCEER-86-1, University of Nevada, Reno, July 1986.
CCEER-86-2	Saiidi, M., R. Lawver, and J. Hart, “User's Manual of ISADAB and SIBA, Computer Programs for Nonlinear Transverse Analysis of Highway Bridges Subjected to Static and Dynamic Lateral Loads,” Civil Engineering Department, Report No. CCEER-86-2, University of Nevada, Reno, September 1986.
CCEER-87-1	Siddharthan, R., “Dynamic Effective Stress Response of Surface and Embedded Footings in Sand,” Civil Engineering Department, Report No. CCEER-86-2, University of Nevada, Reno, June 1987.
CCEER-87-2	Norris, G. and R. Sack, “Lateral and Rotational Stiffness of Pile Groups for Seismic Analysis of Highway Bridges,” Civil Engineering Department, Report No. CCEER-87-2, University of Nevada, Reno, June 1987.
CCEER-88-1	Orie, J. and M. Saiidi, “A Preliminary Study of One-Way Reinforced Concrete Pier Hinges

Subjected to Shear and Flexure," Civil Engineering Department, Report No. CCEER-88-1, University of Nevada, Reno, January 1988.

CCEER-88-2 Orie, D., M. Saiidi, and B. Douglas, "A Micro-CAD System for Seismic Design of Regular Highway Bridges," Civil Engineering Department, Report No. CCEER-88-2, University of Nevada, Reno, June 1988.

CCEER-88-3 Orie, D. and M. Saiidi, "User's Manual for Micro-SARB, a Microcomputer Program for Seismic Analysis of Regular Highway Bridges," Civil Engineering Department, Report No. CCEER-88-3, University of Nevada, Reno, October 1988.

CCEER-89-1 Douglas, B., M. Saiidi, R. Hayes, and G. Holcomb, "A Comprehensive Study of the Loads and Pressures Exerted on Wall Forms by the Placement of Concrete," Civil Engineering Department, Report No. CCEER-89-1, University of Nevada, Reno, February 1989.

CCEER-89-2 Richardson, J. and B. Douglas, "Dynamic Response Analysis of the Dominion Road Bridge Test Data," Civil Engineering Department, Report No. CCEER-89-2, University of Nevada, Reno, March 1989.

CCEER-89-2 Vrontinos, S., M. Saiidi, and B. Douglas, "A Simple Model to Predict the Ultimate Response of R/C Beams with Concrete Overlays," Civil Engineering Department, Report NO. CCEER-89-2, University of Nevada, Reno, June 1989.

CCEER-89-3 Ebrahimpour, A. and P. Jagadish, "Statistical Modeling of Bridge Traffic Loads - A Case Study," Civil Engineering Department, Report No. CCEER-89-3, University of Nevada, Reno, December 1989.

CCEER-89-4 Shields, J. and M. Saiidi, "Direct Field Measurement of Prestress Losses in Box Girder Bridges," Civil Engineering Department, Report No. CCEER-89-4, University of Nevada, Reno, December 1989.

CCEER-90-1 Saiidi, M., E. Maragakis, G. Ghusn, Y. Jiang, and D. Schwartz, "Survey and Evaluation of Nevada's Transportation Infrastructure, Task 7.2 - Highway Bridges, Final Report," Civil Engineering Department, Report No. CCEER 90-1, University of Nevada, Reno, October 1990.

CCEER-90-2 Abdel-Ghaffar, S., E. Maragakis, and M. Saiidi, "Analysis of the Response of Reinforced Concrete Structures During the Whittier Earthquake 1987," Civil Engineering Department, Report No. CCEER 90-2, University of Nevada, Reno, October 1990.

CCEER-91-1 Saiidi, M., E. Hwang, E. Maragakis, and B. Douglas, "Dynamic Testing and the Analysis of the Flamingo Road Interchange," Civil Engineering Department, Report No. CCEER-91-1, University of Nevada, Reno, February 1991.

CCEER-91-2 Norris, G., R. Siddharthan, Z. Zafir, S. Abdel-Ghaffar, and P. Gowda, "Soil-Foundation-Structure Behavior at the Oakland Outer Harbor Wharf," Civil Engineering Department, Report No. CCEER-91-2, University of Nevada, Reno, July 1991.

CCEER-91-3 Norris, G., "Seismic Lateral and Rotational Pile Foundation Stiffnesses at Cypress," Civil Engineering Department, Report No. CCEER-91-3, University of Nevada, Reno, August 1991.

CCEER-91-4 O'Connor, D. and M. Saiidi, "A Study of Protective Overlays for Highway Bridge Decks in Nevada, with Emphasis on Polyester-Styrene Polymer Concrete," Civil Engineering Department, Report No. CCEER-91-4, University of Nevada, Reno, October 1991.

CCEER-91-5 O'Connor, D.N. and M. Saiidi, “Laboratory Studies of Polyester-Styrene Polymer Concrete Engineering Properties,” Civil Engineering Department, Report No. CCEER-91-5, University of Nevada, Reno, November 1991.

CCEER-92-1 Straw, D.L. and M. Saiidi, “Scale Model Testing of One-Way Reinforced Concrete Pier Hinges Subject to Combined Axial Force, Shear and Flexure,” edited by D.N. O'Connor, Civil Engineering Department, Report No. CCEER-92-1, University of Nevada, Reno, March 1992.

CCEER-92-2 Wehbe, N., M. Saiidi, and F. Gordaninejad, “Basic Behavior of Composite Sections Made of Concrete Slabs and Graphite Epoxy Beams,” Civil Engineering Department, Report No. CCEER-92-2, University of Nevada, Reno, August 1992.

CCEER-92-3 Saiidi, M. and E. Hutchens, “A Study of Prestress Changes in A Post-Tensioned Bridge During the First 30 Months,” Civil Engineering Department, Report No. CCEER-92-3, University of Nevada, Reno, April 1992.

CCEER-92-4 Saiidi, M., B. Douglas, S. Feng, E. Hwang, and E. Maragakis, “Effects of Axial Force on Frequency of Prestressed Concrete Bridges,” Civil Engineering Department, Report No. CCEER-92-4, University of Nevada, Reno, August 1992.

CCEER-92-5 Siddharthan, R., and Z. Zafir, “Response of Layered Deposits to Traveling Surface Pressure Waves,” Civil Engineering Department, Report No. CCEER-92-5, University of Nevada, Reno, September 1992.

CCEER-92-6 Norris, G., and Z. Zafir, “Liquefaction and Residual Strength of Loose Sands from Drained Triaxial Tests,” Civil Engineering Department, Report No. CCEER-92-6, University of Nevada, Reno, September 1992.

CCEER-92-6-A Norris, G., Siddharthan, R., Zafir, Z. and Madhu, R. “Liquefaction and Residual Strength of Sands from Drained Triaxial Tests,” Civil Engineering Department, Report No. CCEER-92-6-A, University of Nevada, Reno, September 1992.

CCEER-92-7 Douglas, B., “Some Thoughts Regarding the Improvement of the University of Nevada, Reno's National Academic Standing,” Civil Engineering Department, Report No. CCEER-92-7, University of Nevada, Reno, September 1992.

CCEER-92-8 Saiidi, M., E. Maragakis, and S. Feng, “An Evaluation of the Current Caltrans Seismic Restrainer Design Method,” Civil Engineering Department, Report No. CCEER-92-8, University of Nevada, Reno, October 1992.

CCEER-92-9 O'Connor, D., M. Saiidi, and E. Maragakis, “Effect of Hinge Restrainers on the Response of the Madrone Drive Undercrossing During the Loma Prieta Earthquake,” Civil Engineering Department, Report No. CCEER-92-9, University of Nevada, Reno, February 1993.

CCEER-92-10 O'Connor, D., and M. Saiidi, “Laboratory Studies of Polyester Concrete: Compressive Strength at Elevated Temperatures and Following Temperature Cycling, Bond Strength to Portland Cement Concrete, and Modulus of Elasticity,” Civil Engineering Department, Report No. CCEER-92-10, University of Nevada, Reno, February 1993.

CCEER-92-11 Wehbe, N., M. Saiidi, and D. O'Connor, “Economic Impact of Passage of Spent Fuel Traffic on Two Bridges in Northeast Nevada,” Civil Engineering Department, Report No. CCEER-92-11, University of Nevada, Reno, December 1992.

CCEER-93-1 Jiang, Y., and M. Saiidi, “Behavior, Design, and Retrofit of Reinforced Concrete One-way Bridge Column Hinges,” edited by D. O'Connor, Civil Engineering Department, Report

No. CCEER-93-1, University of Nevada, Reno, March 1993.

CCEER-93-2 Abdel-Ghaffar, S., E. Maragakis, and M. Saiidi, “Evaluation of the Response of the Aptos Creek Bridge During the 1989 Loma Prieta Earthquake,” Civil Engineering Department, Report No. CCEER-93-2, University of Nevada, Reno, June 1993.

CCEER-93-3 Sanders, D.H., B.M. Douglas, and T.L. Martin, “Seismic Retrofit Prioritization of Nevada Bridges,” Civil Engineering Department, Report No. CCEER-93-3, University of Nevada, Reno, July 1993.

CCEER-93-4 Abdel-Ghaffar, S., E. Maragakis, and M. Saiidi, “Performance of Hinge Restrainers in the Huntington Avenue Overhead During the 1989 Loma Prieta Earthquake,” Civil Engineering Department, Report No. CCEER-93-4, University of Nevada, Reno, June 1993 (in final preparation).

CCEER-93-5 Maragakis, E., M. Saiidi, S. Feng, and L. Flournoy, “Effects of Hinge Restrainers on the Response of the San Gregorio Bridge during the Loma Prieta Earthquake,” (in final preparation) Civil Engineering Department, Report No. CCEER-93-5, University of Nevada, Reno.

CCEER-93-6 Saiidi, M., E. Maragakis, S. Abdel-Ghaffar, S. Feng, and D. O'Connor, “Response of Bridge Hinge Restrainers during Earthquakes -Field Performance, Analysis, and Design,” Civil Engineering Department, Report No. CCEER-93-6, University of Nevada, Reno, May 1993.

CCEER-93-7 Wehbe, N., Saiidi, M., Maragakis, E., and Sanders, D., “Adequacy of Three Highway Structures in Southern Nevada for Spent Fuel Transportation,” Civil Engineering Department, Report No. CCEER-93-7, University of Nevada, Reno, August 1993.

CCEER-93-8 Roybal, J., Sanders, D.H., and Maragakis, E., “Vulnerability Assessment of Masonry in the Reno-Carson City Urban Corridor,” Civil Engineering Department, Report No. CCEER-93-8, University of Nevada, Reno, May 1993.

CCEER-93-9 Zafir, Z. and Siddharthan, R., “MOVLOAD: A Program to Determine the Behavior of Nonlinear Horizontally Layered Medium Under Moving Load,” Civil Engineering Department, Report No. CCEER-93-9, University of Nevada, Reno, August 1993.

CCEER-93-10 O'Connor, D.N., Saiidi, M., and Maragakis, E.A., “A Study of Bridge Column Seismic Damage Susceptibility at the Interstate 80/U.S. 395 Interchange in Reno, Nevada,” Civil Engineering Department, Report No. CCEER-93-10, University of Nevada, Reno, October 1993.

CCEER-94-1 Maragakis, E., B. Douglas, and E. Abdelwahed, “Preliminary Dynamic Analysis of a Railroad Bridge,” Report CCEER-94-1, January 1994.

CCEER-94-2 Douglas, B.M., Maragakis, E.A., and Feng, S., “Stiffness Evaluation of Pile Foundation of Cazenovia Creek Overpass,” Civil Engineering Department, Report No. CCEER-94-2, University of Nevada, Reno, March 1994.

CCEER-94-3 Douglas, B.M., Maragakis, E.A., and Feng, S., “Summary of Pretest Analysis of Cazenovia Creek Bridge,” Civil Engineering Department, Report No. CCEER-94-3, University of Nevada, Reno, April 1994.

CCEER-94-4 Norris, G.M., Madhu, R., Valceschini, R., and Ashour, M., “Liquefaction and Residual Strength of Loose Sands from Drained Triaxial Tests,” Report 2, Vol. 1&2, Civil Engineering Department, Report No. CCEER-94-4, University of Nevada, Reno, August 1994.

CCEER-94-5 Saiidi, M., Hutchens, E., and Gardella, D., “Prestress Losses in a Post-Tensioned R/C Box Girder Bridge in Southern Nevada,” Civil Engineering Department, CCEER-94-5, University of Nevada, Reno, August 1994.

CCEER-95-1 Siddharthan, R., El-Gamal, M., and Maragakis, E.A., “Nonlinear Bridge Abutment , Verification, and Design Curves,” Civil Engineering Department, CCEER-95-1, University of Nevada, Reno, January 1995.

CCEER-95-2 Ashour, M. and Norris, G., “Liquefaction and Undrained Response Evaluation of Sands from Drained Formulation,” Civil Engineering Department, Report No. CCEER-95-2, University of Nevada, Reno, February 1995.

CCEER-95-3 Wehbe, N., Saiidi, M., Sanders, D. and Douglas, B., “Ductility of Rectangular Reinforced Concrete Bridge Columns with Moderate Confinement,” Civil Engineering Department, Report No. CCEER-95-3, University of Nevada, Reno, July 1995.

CCEER-95-4 Martin, T.., Saiidi, M. and Sanders, D., “Seismic Retrofit of Column-Pier Cap Connections in Bridges in Northern Nevada,” Civil Engineering Department, Report No. CCEER-95-4, University of Nevada, Reno, August 1995.

CCEER-95-5 Darwish, I., Saiidi, M. and Sanders, D., “Experimental Study of Seismic Susceptibility Column-Footing Connections in Bridges in Northern Nevada,” Civil Engineering Department, Report No. CCEER-95-5, University of Nevada, Reno, September 1995.

CCEER-95-6 Griffin, G., Saiidi, M. and Maragakis, E., “Nonlinear Seismic Response of Isolated Bridges and Effects of Pier Ductility Demand,” Civil Engineering Department, Report No. CCEER-95-6, University of Nevada, Reno, November 1995.

CCEER-95-7 Acharya, S.., Saiidi, M. and Sanders, D., “Seismic Retrofit of Bridge Footings and Column-Footing Connections,” Civil Engineering Department, Report No. CCEER-95-7, University of Nevada, Reno, November 1995.

CCEER-95-8 Maragakis, E., Douglas, B., and Sandirasegaram, U., “Full-Scale Field Resonance Tests of a Railway Bridge,” A Report to the Association of American Railroads, Civil Engineering Department, Report No. CCEER-95-8, University of Nevada, Reno, December 1995.

CCEER-95-9 Douglas, B., Maragakis, E. and Feng, S., “System Identification Studies on Cazenovia Creek Overpass,” Report for the National Center for Earthquake Engineering Research, Civil Engineering Department, Report No. CCEER-95-9, University of Nevada, Reno, October 1995.

CCEER-96-1 El-Gamal, M.E. and Siddharthan, R.V., “Programs to Computer Translational Stiffness of Seat-Type Bridge Abutment,” Civil Engineering Department, Report No. CCEER-96-1, University of Nevada, Reno, March 1996.

CCEER-96-2 Labia, Y., Saiidi, M. and Douglas, B., “Evaluation and Repair of Full-Scale Prestressed Concrete Box Girders,” A Report to the National Science Foundation, Research Grant CMS-9201908, Civil Engineering Department, Report No. CCEER-96-2, University of Nevada, Reno, May 1996.

CCEER-96-3 Darwish, I., Saiidi, M. and Sanders, D., “Seismic Retrofit of R/C Oblong Tapered Bridge Columns with Inadequate Bar Anchorage in Columns and Footings,” A Report to the Nevada Department of Transportation, Civil Engineering Department, Report No. CCEER-96-3, University of Nevada, Reno, May 1996.

CCEER-96-4 Ashour, M., Pilling, R., Norris, G. and Perez, H., “The Prediction of Lateral Load Behavior of Single Piles and Pile Groups Using the Strain Wedge Model," A Report to the California Department of Transportation, Civil Engineering Department, Report No. CCEER-96-4, University of Nevada, Reno, June 1996.

CCEER-97-1-A Rimal, P. and Itani, A. "Sensitivity Analysis of Fatigue Evaluations of Steel Bridges," Center for Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada Report No. CCEER-97-1-A, September, 1997.

CCEER-97-1-B Maragakis, E., Douglas, B., and Sandirasegaram, U. "Full-Scale Field Resonance Tests of a Railway Bridge," A Report to the Association of American Railroads, Civil Engineering Department, University of Nevada, Reno, May, 1996.

CCEER-97-2 Wehbe, N., Saiidi, M., and D. Sanders, "Effect of Confinement and Flares on the Seismic Performance of Reinforced Concrete Bridge Columns," Civil Engineering Department, Report No. CCEER-97-2, University of Nevada, Reno, September 1997.

CCEER-97-3 Darwish, I., M. Saiidi, G. Norris, and E. Maragakis, "Determination of In-Situ Footing Stiffness Using Full-Scale Dynamic Field Testing," A Report to the Nevada Department of Transportation, Structural Design Division, Carson City, Nevada, Report No. CCEER-97-3, University of Nevada, Reno, October 1997.

CCEER-97-4-A Itani, A. "Cyclic Behavior of Richmond-San Rafael Tower Links," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-97-4, August 1997.

CCEER-97-4-B Wehbe, N., and M. Saiidi, "User's Manual for RCMC v. 1.2 : A Computer Program for Moment-Curvature Analysis of Confined and Unconfined Reinforced Concrete Sections," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-97-4, November, 1997.

CCEER-97-5 Isakovic, T., M. Saiidi, and A. Itani, "Influence of new Bridge Configurations on Seismic Performance," Department of Civil Engineering, University of Nevada, Reno, Report No. CCEER-97-5, September, 1997.

CCEER-98-1 Itani, A., Vesco, T. and Dietrich, A., "Cyclic Behavior of "as Built" Laced Members With End Gusset Plates on the San Francisco Bay Bridge," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada Report No. CCEER-98-1, March, 1998.

CCEER-98-2 G. Norris and M. Ashour, "Liquefaction and Undrained Response Evaluation of Sands from Drained Formulation," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-98-2, May, 1998.

CCEER-98-3 Qingbin, Chen, B. M. Douglas, E. Maragakis, and I. G. Buckle, "Extraction of Nonlinear Hysteretic Properties of Seismically Isolated Bridges from Quick-Release Field Tests," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-98-3, June, 1998.

CCEER-98-4 Maragakis, E., B. M. Douglas, and C. Qingbin, "Full-Scale Field Capacity Tests of a Railway Bridge," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-98-4, June, 1998.

CCEER-98-5 Itani, A., Douglas, B., and Woodgate, J., "Cyclic Behavior of Richmond-San Rafael Retrofitted Tower Leg," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno. Report No. CCEER-98-5, June 1998

CCEER-98-6 Moore, R., Saiidi, M., and Itani, A., “Seismic Behavior of New Bridges with Skew and Curvature,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno. Report No. CCEER-98-6, October, 1998.

CCEER-98-7 Itani, A and Dietrich, A, “Cyclic Behavior of Double Gusset Plate Connections,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-98-5, December, 1998.

CCEER-99-1 Caywood, C., M. Saiidi, and D. Sanders, “Seismic Retrofit of Flared Bridge Columns with Steel Jackets,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-1, February 1999.

CCEER-99-2 Mangoba, N., M. Mayberry, and M. Saiidi, “Prestress Loss in Four Box Girder Bridges in Northern Nevada,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-2, March 1999.

CCEER-99-3 Abo-Shadi, N., M. Saiidi, and D. Sanders, “Seismic Response of Bridge Pier Walls in the Weak Direction,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-3, April 1999.

CCEER-99-4 Buzick, A., and M. Saiidi, “Shear Strength and Shear Fatigue Behavior of Full-Scale Prestressed Concrete Box Girders,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-4, April 1999.

CCEER-99-5 Randall, M., M. Saiidi, E. Maragakis and T. Isakovic, “Restrainer Design Procedures For Multi-Span Simply-Supported Bridges,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-5, April 1999.

CCEER-99-6 Wehbe, N. and M. Saiidi, “User's Manual for RCMC v. 1.2, A Computer Program for Moment-Curvature Analysis of Confined and Unconfined Reinforced Concrete Sections,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-6, May 1999.

CCEER-99-7 Burda, J. and A. Itani, “Studies of Seismic Behavior of Steel Base Plates,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-7, May 1999.

CCEER-99-8 Ashour, M. and G. Norris, “Refinement of the Strain Wedge Model Program,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-8, March 1999.

CCEER-99-9 Dietrich, A., and A. Itani, “Cyclic Behavior of Laced and Perforated Steel Members on the San Francisco-Oakland Bay Bridge,” Civil Engineering Department, University, Reno, Report No. CCEER-99-9, December 1999.

CCEER 99-10 Itani, A., A. Dietrich, “Cyclic Behavior of Built Up Steel Members and their Connections,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-10, December 1999.

CCEER 99-10-A Itani, A., E. Maragakis and P. He, “Fatigue Behavior of Riveted Open Deck Railroad Bridge Girders,” Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-99-10-A, August 1999.

CCEER 99-11 Itani, A., J. Woodgate, “Axial and Rotational Ductility of Built Up Structural Steel Members,” Civil Engineering Department, University of Nevada, Reno, Report No.

CCEER-99-11, December 1999.

CCEER-99-12 Sgambelluri, M., Sanders, D.H., and Saiidi, M.S., "Behavior of One-Way Reinforced Concrete Bridge Column Hinges in the Weak Direction," Department of Civil Engineering, University of Nevada, Reno, Report No. CCEER-99-12, December 1999.

CCEER-99-13 Laplace, P., Sanders, D.H., Douglas, B, and Saiidi, M, "Shake Table Testing of Flexure Dominated Reinforced Concrete Bridge Columns", Department of Civil Engineering, University of Nevada, Reno, Report No. CCEER-99-13, December 1999.

CCEER-99-14 Ahmad M. Itani, Jose A. Zepeda, and Elizabeth A. Ware "Cyclic Behavior of Steel Moment Frame Connections for the Moscone Center Expansion," Department of Civil Engineering, University of Nevada, Reno, Report No. CCEER-99-14, December 1999.

CCEER 00-1 Ashour, M., and Norris, G. "Undrained Lateral Pile and Pile Group Response in Saturated Sand," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-1, May 1999. January 2000.

CCEER 00-2 Saiidi, M. and Wehbe, N., "A Comparison of Confinement Requirements in Different Codes for Rectangular, Circular, and Double-Spiral RC Bridge Columns," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-2, January 2000.

CCEER 00-3 McElhaney, B., M. Saiidi, and D. Sanders, "Shake Table Testing of Flared Bridge Columns With Steel Jacket Retrofit," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-3, January 2000.

CCEER 00-4 Martinovic, F., M. Saiidi, D. Sanders, and F. Gordaninejad, "Dynamic Testing of Non-Prismatic Reinforced Concrete Bridge Columns Retrofitted with FRP Jackets," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-4, January 2000.

CCEER 00-5 Itani, A., and M. Saiidi, "Seismic Evaluation of Steel Joints for UCLA Center for Health Science Westwood Replacement Hospital," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-5, February 2000.

CCEER 00-6 Will, J. and D. Sanders, "High Performance Concrete Using Nevada Aggregates," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-6, May 2000.

CCEER 00-7 French, C., and M. Saiidi, "A Comparison of Static and Dynamic Performance of Models of Flared Bridge Columns," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER-00-7, October 2000.

CCEER 00-8 Itani, A., H. Sedarat, "Seismic Analysis of the AISI LRFD Design Example of Steel Highway Bridges," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 00-08, November 2000.

CCEER 00-9 Moore, J., D. Sanders, and M. Saiidi, "Shake Table Testing of 1960's Two Column Bent with Hinges Bases," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 00-09, December 2000.

CCEER 00-10 Asthana, M., D. Sanders, and M. Saiidi, "One-Way Reinforced Concrete Bridge Column Hinges in the Weak Direction," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 00-10, April 2001.

CCEER 01-1 Ah Sha, H., D. Sanders, M. Saiidi, "Early Age Shrinkage and Cracking of Nevada Concrete Bridge Decks," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-01, May 2001.

CCEER 01-2 Ashour, M. and G. Norris, "Pile Group program for Full Material Modeling a Progressive Failure," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-02, July 2001.

CCEER 01-3 Itani, A., C. Lanaud, and P. Dusicka, "Non-Linear Finite Element Analysis of Built-Up Shear Links," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-03, July 2001.

CCEER 01-4 Saiidi, M., J. Mortensen, and F. Martinovic, "Analysis and Retrofit of Fixed Flared Columns with Glass Fiber-Reinforced Plastic Jacketing," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-4, August 2001

CCEER 01-5 Not Published

CCEER 01-6 Laplace, P., D. Sanders, and M. Saiidi, "Experimental Study and Analysis of Retrofitted Flexure and Shear Dominated Circular Reinforced Concrete Bridge Columns Subjected to Shake Table Excitation," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-6, June 2001.

CCEER 01-7 Reppi, F., and D. Sanders, "Removal and Replacement of Cast-in-Place, Post-tensioned, Box Girder Bridge," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 01-7, December 2001.

CCEER 02-1 Pulido, C., M. Saiidi, D. Sanders, and A. Itani, "Seismic Performance and Retrofitting of Reinforced Concrete Bridge Bents," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-1, January 2002.

CCEER 02-2 Yang, Q., M. Saiidi, H. Wang, and A. Itani, "Influence of Ground Motion Incoherency on Earthquake Response of Multi-Support Structures," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-2, May 2002.

CCEER 02-3 M. Saiidi, B. Gopalakrishnan, E. Reinhardt, and R. Siddharthan, "A Preliminary Study of Shake Table Response of A Two-Column Bridge Bent on Flexible Footings," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-03, June 2002.

CCEER 02-4 Not Published

CCEER 02-5 Banghart, A., Sanders, D., Saiidi, M., "Evaluation of Concrete Mixes for Filling the Steel Arches in the Galena Creek Bridge," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-05, June 2002.

CCEER 02-6 Dusicka, P., Itani, A., Buckle, I. G., "Cyclic Behavior of Shear Links and Tower Shaft Assembly of San Francisco – Oakland Bay Bridge Tower," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-06, July 2002.

CCEER 02-7 Mortensen, J., and M. Saiidi, "A Performance-Based Design Method for Confinement in Circular Columns," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-07, November 2002.

CCEER 03-1 Wehbe, N., and M. Saiidi, "User's manual for SPMC v. 1.0 : A Computer Program for Moment-Curvature Analysis of Reinforced Concrete Sections with Interlocking Spirals," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-03-1, May, 2003.

CCEER 03-2 Wehbe, N., and M. Saiidi, "User's manual for RCMC v. 2.0 : A Computer Program for Moment-Curvature Analysis of Confined and Unconfined Reinforced Concrete Sections," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-03-2, June, 2003.

CCEER 03-3 Nada, H., D. Sanders, and M. Saiidi, "Seismic Performance of RC Bridge Frames with Architectural-Flared Columns," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 03-3, January 2003.

CCEER 03-4 Reinhardt, E., M. Saiidi, and R. Siddharthan, "Seismic Performance of a CFRP/ Concrete Bridge Bent on Flexible Footings," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 03-4, August 2003.

CCEER 03-5 Johnson, N., M. Saiidi, A. Itani, and S. Ladkany, "Seismic Retrofit of Octagonal Columns with Pedestal and One-Way Hinge at the Base," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, and Report No. CCEER-03-5, August 2003.

CCEER 03-6 Mortensen, C., M. Saiidi, and S. Ladkany, "Creep and Shrinkage Losses in Highly Variable Climates," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Report No. CCEER-03-6, September 2003.

CCEER 03- 7 Ayoub, C., M. Saiidi, and A. Itani, "A Study of Shape-Memory-Alloy-Reinforced Beams and Cubes," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-03-7, October 2003.

CCEER 03-8 Chandane, S., D. Sanders, and M. Saiidi, "Static and Dynamic Performance of RC Bridge Bents with Architectural-Flared Columns," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-03-8, November 2003.

CCEER 04-1 Olaegbe, C., and Saiidi, M., "Effect of Loading History on Shake Table Performance of A Two-Column Bent with Infill Wall," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-1, January 2004.

CCEER 04-2 Johnson, R., Maragakis, E., Saiidi, M., and DesRoches, R., "Experimental Evaluation of Seismic Performance of SMA Bridge Restrainers," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-2, February 2004.

CCEER 04-3 Moustafa, K., Sanders, D., and Saiidi, M., "Impact of Aspect Ratio on Two-Column Bent Seismic Performance," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-3, February 2004.

CCEER 04-4 Maragakis, E., Saiidi, M., Sanchez-Camargo, F., and Elfass, S., "Seismic Performance of Bridge Restrainers At In-Span Hinges," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-4, March 2004.

CCEER 04-5 Ashour, M., Norris, G. and Elfass, S., “Analysis of Laterally Loaded Long or Intermediate Drilled Shafts of Small or Large Diameter in Layered Soil,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-5, June 2004.

CCEER 04-6 Correal, J., Saiidi, M. and Sanders, D., “Seismic Performance of RC Bridge Columns Reinforced with Two Interlocking Spirals,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-6, August 2004.

CCEER 04-7 Dusicka, P., Itani, A. and Buckle, I., “Cyclic Response and Low Cycle Fatigue Characteristics of Plate Steels,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-7, November 2004.

CCEER 04-8 Dusicka, P., Itani, A. and Buckle, I., “Built-up Shear Links as Energy Dissipaters for Seismic Protection of Bridges,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-8, November 2004.

CCEER 04-9 Sureshkumar, K., Saiidi, S., Itani, A. and Ladkany, S., “Seismic Retrofit of Two-Column Bents with Diamond Shape Columns,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-04-9, November 2004.

CCEER 05-1 Wang, H. and Saiidi, S., “A Study of RC Columns with Shape Memory Alloy and Engineered Cementitious Composites,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-1, January 2005.

CCEER 05-2 Johnson, R., Saiidi, S. and Maragakis, E., “A Study of Fiber Reinforced Plastics for Seismic Bridge Restrainers,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-2, January 2005.

CCEER 05-3 Carden, L.P., Itani, A.M., Buckle, I.G, “Seismic Load Path in Steel Girder Bridge Superstructures,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-3, January 2005.

CCEER 05-4 Carden, L.P., Itani, A.M., Buckle, I.G, “Seismic Performance of Steel Girder Bridge Superstructures with Ductile End Cross Frames and Seismic Isolation,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-4, January 2005.

CCEER 05-5 Goodwin, E., Maragakis, M., Itani, A. and Luo, S., “Experimental Evaluation of the Seismic Performance of Hospital Piping Subassemblies,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-5, February 2005.

CCEER 05-6 Zadeh M. S., Saiidi, S, Itani, A. and Ladkany, S., “Seismic Vulnerability Evaluation and Retrofit Design of Las Vegas Downtown Viaduct,” Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-6, February 2005.

CCEER 05-7 Phan, V., Saiidi, S. and Anderson, J., "Near Fault (Near Field) Ground Motion Effects on Reinforced Concrete Bridge Columns," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-7, August 2005.

CCEER 05-8 Carden, L., Itani, A. and Laplace, P., "Performance of Steel Props at the UNR Fire Science Academy subjected to Repeated Fire," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-8, August 2005.

CCEER 05-9 Yamashita, R. and Sanders, D., "Shake Table Testing and an Analytical Study of Unbonded Prestressed Hollow Concrete Column Constructed with Precast Segments," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-9, August 2005.

CCEER 05-10 Not Published

CCEER 05-11 Carden, L., Itani., A., and Peckan, G., "Recommendations for the Design of Beams and Posts in Bridge Falsework," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-05-11, October 2005.

CCEER 06-01 Cheng, Z., Saiidi, M., and Sanders, D., "Development of a Seismic Design Method for Reinforced Concrete Two-Way Bridge Column Hinges," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-06-01, February 2006.

CCEER 06-02 Johnson, N., Saiidi, M., and Sanders, D., "Large-Scale Experimental and Analytical Studies of a Two-Span Reinforced Concrete Bridge System," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-06-02, March 2006.

CCEER 06-03 Saiidi, M., Ghasemi, H. and Tiras, A., "Seismic Design and Retrofit of Highway Bridges," Proceedings, Second US-Turkey Workshop, Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-06-03, May 2006.

CCEER 07-01 O'Brien, M., Saiidi, M. and Sadrossadat-Zadeh, M., "A Study of Concrete Bridge Columns Using Innovative Materials Subjected to Cyclic Loading," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-01, January 2007.

CCEER 07-02 Sadrossadat-Zadeh, M. and Saiidi, M., "Effect of Strain rate on Stress-Strain Properties and Yield Propagation in Steel Reinforcing Bars," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-02, January 2007.

CCEER 07-03 Sadrossadat-Zadeh, M. and Saiidi, M., "Analytical Study of NEESR-SG 4-Span Bridge Model Using OpenSees," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-03, January 2007.

CCEER 07-04 Nelson, R., Saiidi, M. and Zadeh, S., "Experimental Evaluation of Performance of Conventional Bridge Systems," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-04, October 2007.

CCEER 07-05 Bahen, N. and Sanders, D., "Strut-and-Tie Modeling for Disturbed Regions in Structural Concrete Members with Emphasis on Deep Beams," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-05, December 2007.

CCEER 07-06 Choi, H., Saiidi, M. and Somerville, P., "Effects of Near-Fault Ground Motion and Fault-Rupture on the Seismic Response of Reinforced Concrete Bridges," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-06, December 2007.

CCEER 07-07 Ashour M. and Norris, G., "Report and User Manual on Strain Wedge Model Computer Program for Files and Large Diameter Shafts with LRFD Procedure," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-07-07, October 2007.

CCEER 08-01 Doyle, K. and Saiidi, M., "Seismic Response of Telescopic Pipe Pin Connections," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-08-01, February 2008.

CCEER 08-02 Taylor, M. and Sanders, D., "Seismic Time History Analysis and Instrumentation of the Galena Creek Bridge," Center for Civil Engineering Earthquake Research, Department of Civil Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-08-02, April 2008.

CCEER 08-03 Abdel-Mohti, A. and Pekcan, G., "Seismic Response Assessment and Recommendations for the Design of Skewed Post-Tensioned Concrete Box-Girder Highway Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-08-03, September 2008.

CCEER 08-04 Saiidi, M., Ghasemi, H. and Hook, J., "Long Term Bridge Performance Monitoring, Assessment & Management," Proceedings, FHWA/NSF Workshop on Future Directions," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER 08-04, September 2008.

CCEER 09-01 Brown, A., and Saiidi, M., "Investigation of Near-Fault Ground Motion Effects on Substandard Bridge Columns and Bents," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-09-01, July 2009.

CCEER 09-02 Linke, C., Pekcan, G., and Itani, A., "Detailing of Seismically Resilient Special Truss Moment Frames," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-09-02, August 2009.

CCEER 09-03 Hillis, D., and Saiidi, M., "Design, Construction, and Nonlinear Dynamic Analysis of Three Bridge Bents Used in a Bridge System Test," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-09-03, August 2009.

CCEER 09-04 Bahrami, H., Itani, A., and Buckle, I., "Guidelines for the Seismic Design of Ductile End Cross Frames in Steel Girder Bridge Superstructures," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-09-04, September 2so009.

CCEER 10-01 Zaghi, A. E., and Saiidi, M., "Seismic Design of Pipe-Pin Connections in Concrete Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-01, January 2010.

CCEER 10-02 Pooranampillai, S., Elfass, S., and Norris, G., "Laboratory Study to Assess Load Capacity Increase of Drilled Shafts through Post Grouting," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-02, January 2010.

CCEER 10-03 Itani, A., Grubb, M., and Monzon, E, "Proposed Seismic Provisions and Commentary for Steel Plate Girder Superstructures," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-03, June 2010.

CCEER 10-04 Cruz-Noguez, C., Saiidi, M., "Experimental and Analytical Seismic Studies of a Four-Span Bridge System with Innovative Materials," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-04, September 2010.

CCEER 10-05 Vosooghi, A., Saiidi, M., "Post-Earthquake Evaluation and Emergency Repair of Damaged RC Bridge Columns Using CFRP Materials," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-05, September 2010.

CCEER 10-06 Ayoub, M., Sanders, D., "Testing of Pile Extension Connections to Slab Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-06, October 2010.

CCEER 10-07 Builes-Mejia, J. C. and Itani, A., "Stability of Bridge Column Rebar Cages during Construction," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-07, November 2010.

CCEER 10-08 Monzon, E.V., "Seismic Performance of Steel Plate Girder Bridges with Integral Abutments," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-10-08, November 2010.

CCEER 11-01 Motaref, S., Saiidi, M., and Sanders, D., "Seismic Response of Precast Bridge Columns with Energy Dissipating Joints," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-11-01, May 2011.

CCEER 11-02 Harrison, N. and Sanders, D., "Preliminary Seismic Analysis and Design of Reinforced Concrete Bridge Columns for Curved Bridge Experiments," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-11-02, May 2011.

CCEER 11-03 Vallejera, J. and Sanders, D., "Instrumentation and Monitoring the Galena Creek Bridge," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-11-03, September 2011.

CCEER 11-04 Levi, M., Sanders, D., and Buckle, I., “Seismic Response of Columns in Horizontally Curved Bridges,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-11-04, December 2011.

CCEER 12-01 Saiidi, M., “NSF International Workshop on Bridges of the Future – Wide Spread Implementation of Innovation,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-12-01, January 2012.

CCEER 12-02 Larkin, A.S., Sanders, D., and Saiidi, M., “Unbonded Prestressed Columns for Earthquake Resistance,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-12-02, January 2012.

CCEER 12-03 Arias-Acosta, J. G., Sanders, D., “Seismic Performance of Circular and Interlocking Spirals RC Bridge Columns under Bidirectional Shake Table Loading Part 1,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-12-03, September 2012.

CCEER 12-04 Cukrov, M.E., Sanders, D., “Seismic Performance of Prestressed Pile-To-Bent Cap Connections,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-12-04, September 2012.

CCEER 13-01 Carr, T. and Sanders, D., “Instrumentation and Dynamic Characterization of the Galena Creek Bridge,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-01, January 2013.

CCEER 13-02 Vosooghi, A. and Buckle, I., “Evaluation of the Performance of a Conventional Four-Span Bridge During Shake Table Tests,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-02, January 2013.

CCEER 13-03 Amirihormozaki, E. and Pekcan, G., “Analytical Fragility Curves for Horizontally Curved Steel Girder Highway Bridges,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-03, February 2013.

CCEER 13-04 Almer, K. and Sanders, D., “Longitudinal Seismic Performance of Precast Bridge Girders Integrally Connected to a Cast-in-Place Bentcap,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-04, April 2013.

CCEER 13-05 Monzon, E.V., Itani, A.I., and Buckle, I.G., “Seismic Modeling and Analysis of Curved Steel Plate Girder Bridges,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-05, April 2013.

CCEER 13-06 Monzon, E.V., Buckle, I.G., and Itani, A.I., “Seismic Performance of Curved Steel Plate Girder Bridges with Seismic Isolation,” Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-06, April 2013.

CCEER 13-07 Monzon, E.V., Buckle, I.G., and Itani, A.I., "Seismic Response of Isolated Bridge Superstructure to Incoherent Ground Motions," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-07, April 2013.

CCEER 13-08 Haber, Z.B., Saiidi, M.S., and Sanders, D.H., "Precast Column-Footing Connections for Accelerated Bridge Construction in Seismic Zones," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-08, April 2013.

CCEER 13-09 Ryan, K.L., Coria, C.B., and Dao, N.D., "Large Scale Earthquake Simulation of a Hybrid Lead Rubber Isolation System Designed under Nuclear Seismicity Considerations," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-09, April 2013.

CCEER 13-10 Wibowo, H., Sanford, D.M., Buckle, I.G., and Sanders, D.H., "The Effect of Live Load on the Seismic Response of Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-10, May 2013.

CCEER 13-11 Sanford, D.M., Wibowo, H., Buckle, I.G., and Sanders, D.H., "Preliminary Experimental Study on the Effect of Live Load on the Seismic Response of Highway Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-11, May 2013.

CCEER 13-12 Saad, A.S., Sanders, D.H., and Buckle, I.G., "Assessment of Foundation Rocking Behavior in Reducing the Seismic Demand on Horizontally Curved Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-12, June 2013.

CCEER 13-13 Ardakani, S.M.S. and Saiidi, M.S., "Design of Reinforced Concrete Bridge Columns for Near-Fault Earthquakes," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-13, July 2013.

CCEER 13-14 Wei, C. and Buckle, I., "Seismic Analysis and Response of Highway Bridges with Hybrid Isolation," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-14, August 2013.

CCEER 13-15 Wibowo, H., Buckle, I.G., and Sanders, D.H., "Experimental and Analytical Investigations on the Effects of Live Load on the Seismic Performance of a Highway Bridge," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-15, August 2013.

CCEER 13-16 Itani, A.M., Monzon, E.V., Grubb, M., and Amirihormozaki, E. "Seismic Design and Nonlinear Evaluation of Steel I-Girder Bridges with Ductile End Cross-Frames," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-16, September 2013.

CCEER 13-17 Kavianipour, F. and Saiidi, M.S., "Experimental and Analytical Seismic Studies of a Four-span Bridge System with Composite Piers," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-17, September 2013.

CCEER 13-18 Mohebbi, A., Ryan, K., and Sanders, D., "Seismic Response of a Highway Bridge with Structural Fuses for Seismic Protection of Piers," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-18, December 2013.

CCEER 13-19 Guzman Pujols, Jean C., Ryan, K.L., "Development of Generalized Fragility Functions for Seismic Induced Content Disruption," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-19, December 2013.

CCEER 14-01 Salem, M. M. A., Pekcan, G., and Itani, A., "Seismic Response Control Of Structures Using Semi-Active and Passive Variable Stiffness Devices," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-01, May 2014.

CCEER 14-02 Saini, A. and Saiidi, M., "Performance-Based Probabilistic Damage Control Approach for Seismic Design of Bridge Columns," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-02, May 2014.

CCEER 14-03 Saini, A. and Saiidi, M., "Post Earthquake Damage Repair of Various Reinforced Concrete Bridge Components," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-03, May 2014.

CCEER 14-04 Monzon, E.V., Itani, A.M., and Grubb, M.A., "Nonlinear Evaluation of the Proposed Seismic Design Procedure for Steel Bridges with Ductile End Cross Frames," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-04, July 2014.

CCEER 14-05 Nakashoji, B. and Saiidi, M.S., "Seismic Performance of Square Nickel-Titanium Reinforced ECC Columns with Headed Couplers," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-05, July 2014.

CCEER 14-06 Tazarv, M. and Saiidi, M.S., "Next Generation of Bridge Columns for Accelerated Bridge Construction in High Seismic Zones," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-06, August 2014.

CCEER 14-07 Mehrsoroush, A. and Saiidi, M.S., "Experimental and Analytical Seismic Studies of Bridge Piers with Innovative Pipe Pin Column-Footing Connections and Precast Cap Beams," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-14-07, December 2014.

CCEER 15-01 Dao, N.D. and Ryan, K.L., "Seismic Response of a Full-scale 5-story Steel Frame Building Isolated by Triple Pendulum Bearings under 3D Excitations," Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-01, January 2015.

CCEER 15-02 Allen, B.M. and Sanders, D.H., “Post-Tensioning Duct Air Pressure Testing Effects on Web Cracking,” Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-02, January 2015.

CCEER 15-03 Akl, A. and Saiidi, M.S., “Time-Dependent Deflection of In-Span Hinges in Prestressed Concrete Box Girder Bridges,” Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-03, May 2015.

CCEER 15-04 Zargar Shotorbani, H. and Ryan, K., “Analytical and Experimental Study of Gap Damper System to Limit Seismic Isolator Displacements in Extreme Earthquakes,” Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-04, June 2015.

CCEER 15-05 Wieser, J., Maragakis, E.M., and Buckle, I., “Experimental and Analytical Investigation of Seismic Bridge-Abutment Interaction in a Curved Highway Bridge,” Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-05, July 2015.

CCEER 15-06 Tazarv, M. and Saiidi, M.S., “Design and Construction of Precast Bent Caps with Pocket Connections for High Seismic Regions,” Center For Civil Engineering Earthquake Research, Department Of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-15-06, August 2015.