A P P E N D I C E S

NCHRP 12-105

App. A – Survey of State Departments of Transportation

App. B – Proposed AASHTO/ASTM Standard Test Method for Bar Couplers

App. C – Proposed AASHTO Specifications for ABC Columns

App. D – Detailed Design Examples

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APPEBDIX D

Detailed Design Examples

Five design examples were developed to illustrate the ABC column connections that were included in the proposed specifications of Appendix C. Table D-1 lists the connection types and the design methods used for each example. Example 1 is a conventional cast-in-place two-span bridge carrying two lanes of traffic designed based on the AASHTO SGS and serves as a benchmark. The superstructure consists of I-girders supported on a drop cap bent and seat-type abutments. Examples 2 to 5 illustrate the design of various connection types consisting of three mechanical splice types, grouted ducts, and pockets (sockets). Grouted sleeves are used in Example 2. The splices are embedded within the columns at the bottom, but are embedded in the cap beam rather than the columns at the top. This is done to avoid dowels protruding from the bottom of the precast cap beam and facilitate construction. Design of swaged couplers is illustrated in Example 3. The couplers are embedded within the columns at both the top and bottom. In this case protrusion of dowels from the bottom of the cap beam is inevitable because swaging process requires access to individual column longitudinal bars and the splices cannot be incorporated within the cap beam. Grouted duct connections are used at the top and headed bar couplers are used at the base of the columns in Example 4. The illustration of design of grouted ducts at the column top provides sufficient detail for their design at the column base. Headed bar couplers are used only at column bases because their use at the column top is cumbersome as it requires dowels extending from the bottom of the cap beam and tight tolerance to line up the dowels with column bars. Design of pocket connections for fully precast columns is illustrated in Example 5, in which pockets are used both at the top and bottom connections.

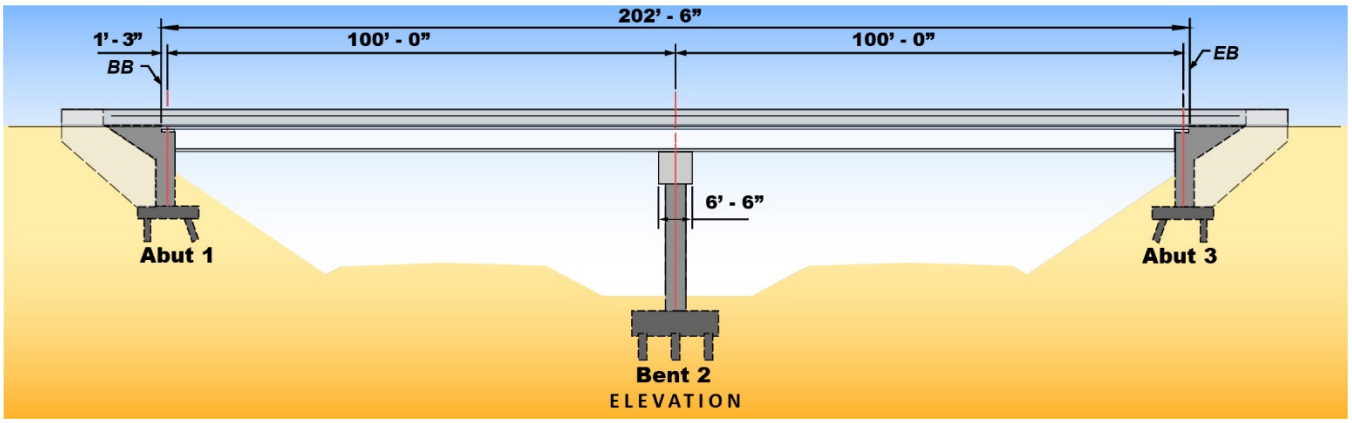
Table D-1. Detailed ABC Column Design Examples

Example No.	Column Connection Type		Design Method
	Top	Bottom	
1	Standard moment connection	Standard moment connection	Benchmark conventional bridge designed per AASHTO SGS
2	Grouted sleeve embedded in cap beam	Grouted sleeve embedded in columns	Designed based on specs developed in NCHRP 12-105
3	Swaged couplers embedded in columns	Swaged couplers embedded in columns	Designed based on specs developed in NCHRP 12-105
4	Grouted ducts in cap beam	Headed bar couplers; two on each longitudinal bar	Designed based on specs developed in NCHRP 12-105
5	Pocket	Pocket	Designed based on specs developed in NCHRP 12-105

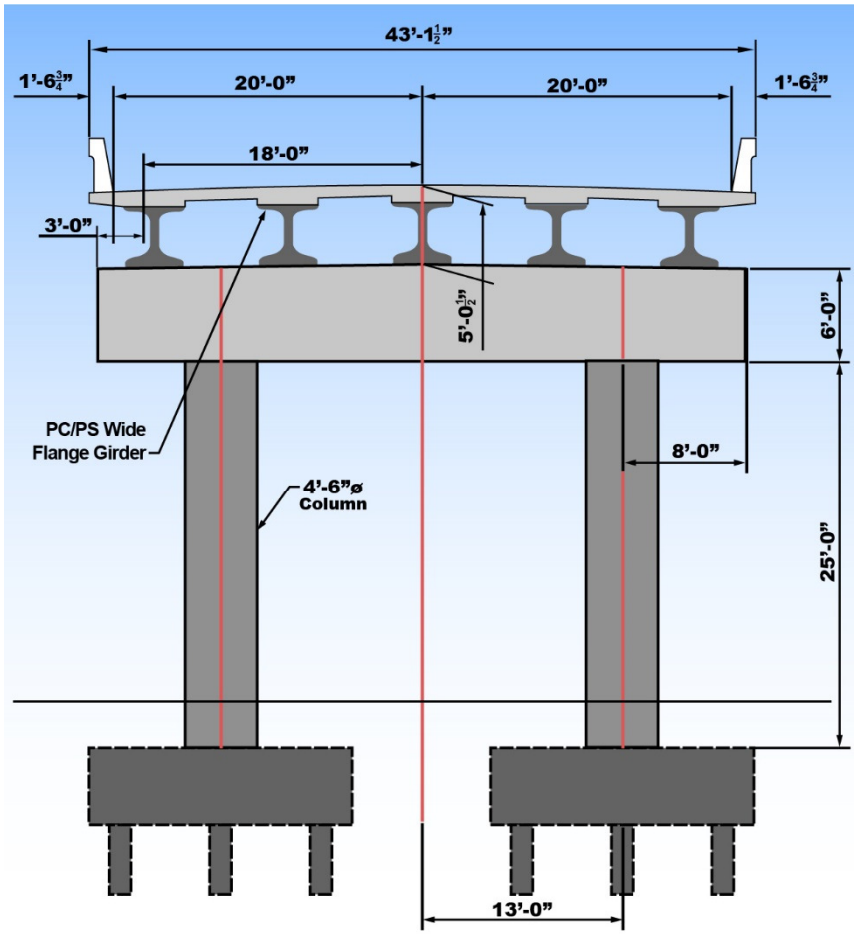
SEISMIC DESIGN EXAMPLE NO. 1

CONVENTIONAL CONSTRUCTION

Seismic Design Example No. 1
Conventional construction



ELEVATION



TYPICAL SECTION

References:

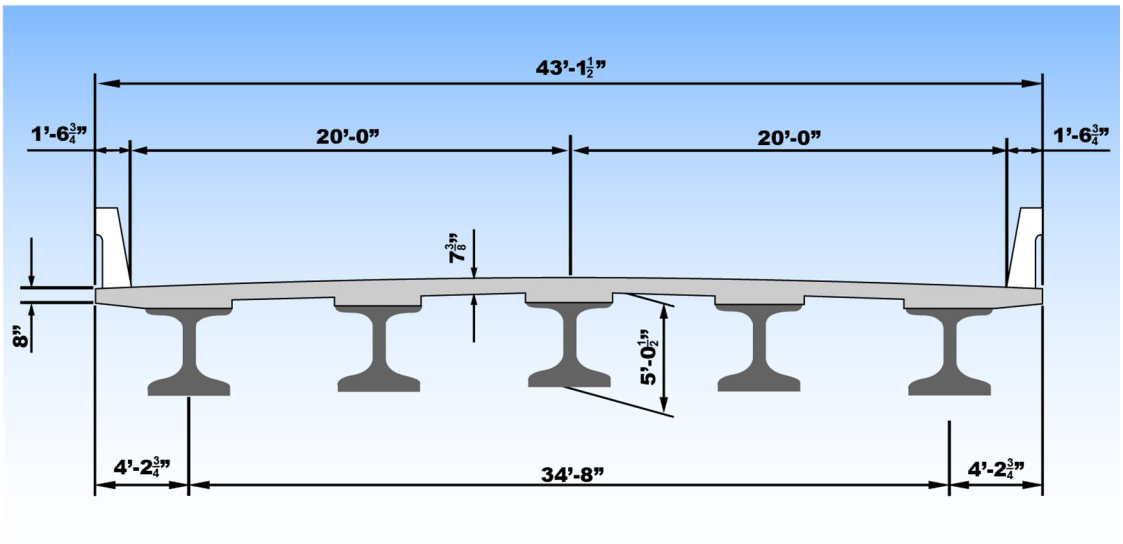
1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011 with 2012, 2014 and 2015 interims (GS)
2. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (AASHTO LRFD)

Material Properties

Element	f_C	f_{CE}	E_{CE}
	ksi	ksi	ksi
Bridge Deck	4.0	5.2	4,347
PC/PS Girders	4.0	5.2	4,347
Bent Cap	4.0	5.2	4,347
Columns	4.0	5.2	4,347

$f_{CE} = 1.3f_C$ *GS Eqn 8.4.4-1*
$w_C = 0.145$ kcf *AASHTO LRFD Table 3.5.1-1*
$K_1 = 1.0$ *AASHTO LRFD 5.4.2.4*
$E_C = 120{,}000\ K_1\ w_C^{2.0} f_C^{0.33}$ *AASHTO LRFD Eqn 5.4.2.4-1*

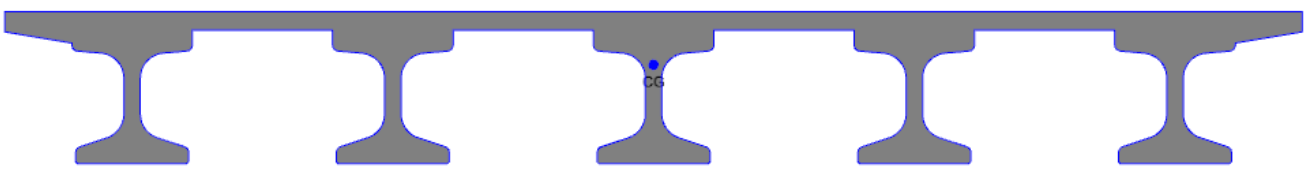
Superstructure



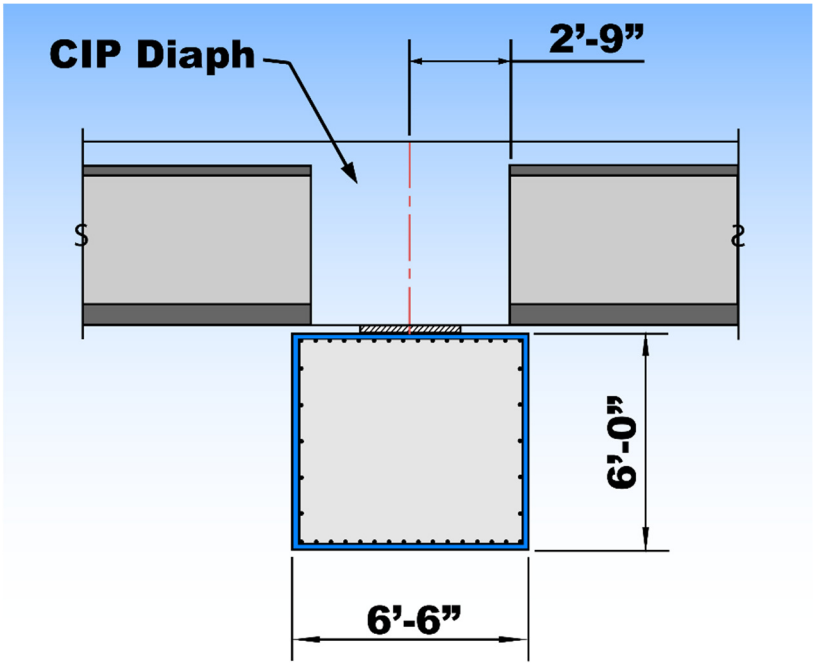
TYPICAL SECTION

Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Superstructure	66.71	205.8	10,455	564,515	1.76	66.71	205.8	10,455	112,903

$I_{EFF}/I_G = 1.0$ for prestressed superstructure
$J_{EFF}/J_G = 0.20$ *GS Eqn 5.6.5-1*
Beta = 90 deg



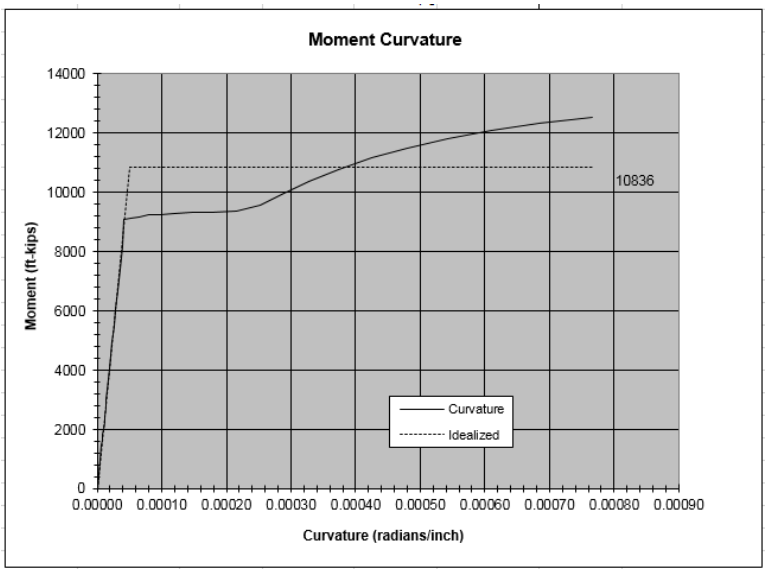
Bent Cap



Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Bent Cap	39.00	117.0	137.3	213.7	3.00	39.00	30.245	1373	42.74

Determine effective moment of inertia and plastic moment using moment curvature analysis and unconfined concrete

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
I_{XEFF} = 30.245 ft^4 I_{XEFF} = 0.259 I_G
Use16-#11 top and bottom ASTM A706 Grade 60
f_{CE} = 5.2 ksi *GS Eqn 8.4.4-1*
f_{YE} = 68.0 ksi *GS Table 8.4.2-14*
ε_{CU}= 0.005



Use 10x I_Y as bent cap is restrained by superstructure

Effective I_Y = 10(137.3) = 1373 ft^4

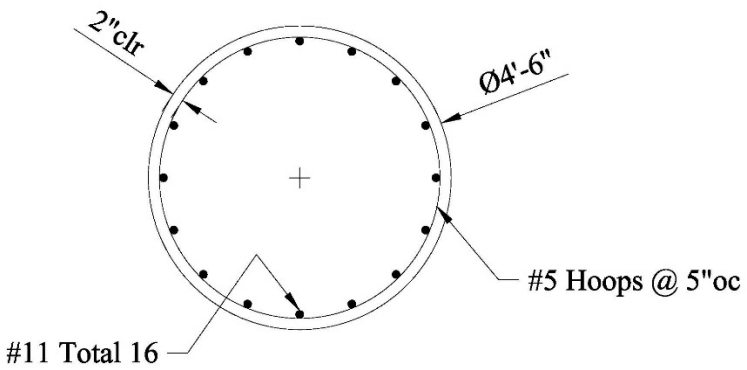
Effective J = 0.20(213.7) = 42.74 ft^4 *GS 5.6.5-1*

M_{NE} = 8,904 ft-kip from moment curvature analysis

Beta = 90 deg

Minimum Cap Width = D_C + 2.0 = 6.50 ft
GS 8.13.4.1

Column



Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Column	15.902	20.129	20.129	40.245	2.25	15.902	6.454	6.454	8.05

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
$I_{XEFF} = I_{YEFF} = 6.454\ ft^4$ $I_{EFF} = 0.321\ I_G$ for DL = 921 kip

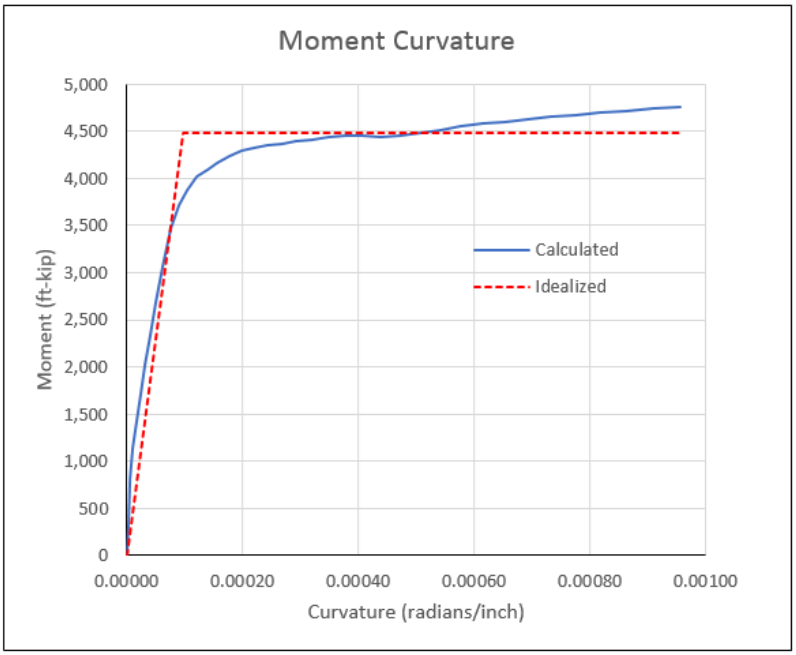
Use16-#11 vertical bars at radius = 23.67 inch ASTM A706 Grade 60 $\rho = 1.09\%$
$f_{CE} = 5.2$ ksi *GS 8.4.4-1*
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi *GS Table 8.4.2-1*
$\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0115$ $\varepsilon^R_{SU} = 0.060$

Use confined concrete within core using Mander's stress-strain model. Use #5 hoops @ 5" oc ASTM A706 Grade 60
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi $\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0150$ $\varepsilon^R_{SU} = 0.090$
Use unconfined concrete outside hoops with $\varepsilon_{CO} = 0.002$ and $\varepsilon_{SP} = 0.005$
Use Axial Load = 921 kip dead load for properties in global model

Effective J = 0.20(40.245) = 8.05 ft^4 *GS 5.6.5-1*

Idealized
$M_P = 4{,}482$ ft-kip
$\phi_Y = 0.000097$ rad/in
$\phi_P = 0.000858$ rad/in
$\phi_U = 0.000955$ rad/in

$M_{NE} = 4{,}352$ ft-kip

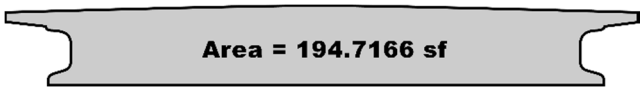


Dead Loads

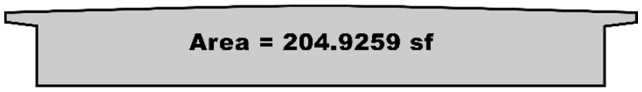
Element	Load Case	No	Area ft^2	w_{DL} klf	Length Feet	DL kip	Comments
Superstructure	SW	1	66.71	10.00	202.5	2026	
Bent Cap	SW	1	39.00	5.85	42.0	246	
Column	SW	2	15.902	2.385	25.0	119	
Total Self Wt						2391	
Barriers	DC	2	3.715	1.11	202.5	225	
Abutment Diaphragm	DC	2	138.2	20.73	2.5	104	
Bent Cap Diaphragm	DC	1	128.0	19.20	5.5	106	
Total Diaphragm						210	
Added DL						434	
Total DL						2825	

Assume no wearing surface and no utilities
Net Area Bent Cap Diaphragm = 194.72-66.71 = 128.0 ft^2
Net Area Abutment Diaphragm = 204.93-66.71 = 138.2 ft^2

SW = Self Weight
DC = DL components/attachments



Bent Cap Diaphragm



Abutment Diaphragm

Statics Check Information

Result Case Name	Applied DY K
Added DL	-434
Barrier	-225
Diaph	-209
Self Wt	-2391
Total DL	-2825

Live Loads
Not considered

Load Combinations
Extreme Event I *AASHTO LRFD Table 3.4.1-1*

Load and Resistance Factors
$\phi = 1.0$ $\gamma_{EQ} = 0.0$ $\gamma_D = 1.0$ *GS 3.70*

Bent Foundation Stiffness

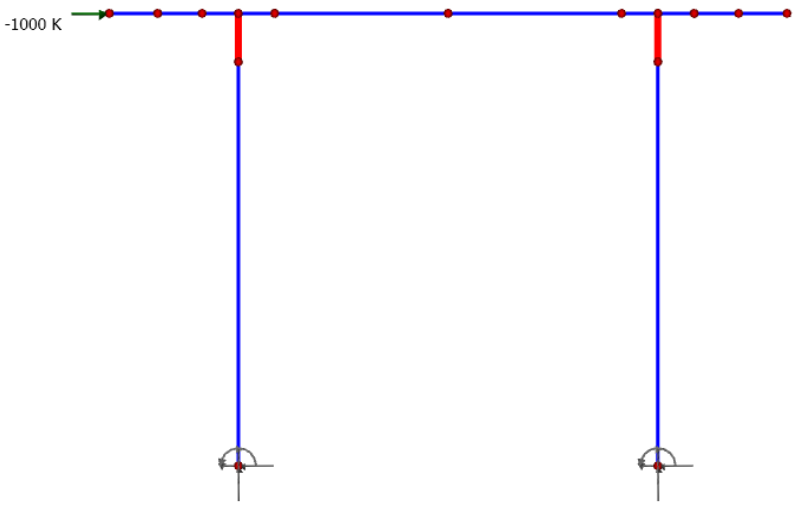
Assume bent foundation piles are in competent material such that foundation flexibility contributes less than 20% to foundation displacement. Soil springs are not required per GS Table 5.3.1-1

Transverse Abutment Springs

Use 50% of Bent Stiffness
in accordance with Caltrans SDC 7.8.2
Δ = 2.653 inch

Bent Stiffness = 1000/2.653 =
Bent Stiffness = 377 kip/in

Transverse Abutment Stiffness = 189 kip/in



Bent Model

Longitudinal Abutment Springs

Use Earthquake-Resisting System (ERS) with Abutment Contribution (Case 2) *GS 5.2.3.2*

Height of Backwall H_W = 60.5 structure depth + 1.50 bearing = 62.0 inch = 5.17 ft
Width of Backwall W_W = 38.75 ft between shear keys

Assume dense sand cohesionless soil

Passive Pressure = $2H_W/3$ = (2)(5.17)/3 = 3.45 ksf *GS Section 5.2.3.3.1*
$P_P = p_P H_W W_W$ = (3.45)(5.17)(38.75) = 690 kip *GS Eqn 5.2.3.3-1*

D_G = Width of Gap between Superstructure and Backwall at Seat Abutment = 1.0 inch
F_W = 0.01 *AASHTO LRFD Table C3.11.1-1*

$$K_{eff1} = \frac{P_p}{(F_w H_W + D_G)} = \frac{690}{[(0.01)(5.17) + 0.083]} = 5114\ kip/ft$$

Column Stiffness = 407 kip/ft each column Based upon Cantilever L = 31.0 ft E_C = 4,347 ksi I_{CR} = 6.454 ft^4
Combined Stiffness = 5,943 kip/ft
Using W = 2,825 kips results in T = 0.764 sec Displacement = 5.56 inch
ARS Force = 2,744 kips Spring Force = 2,367 kip Spring Stiffness = 1,491 kip/ft

Iterate spring stiffness in accordance with GS 5.2.3.3.2 and use K = 1,000 kip/ft
Combined Stiffness = 1,814 kip/ft
Using W = 2,825 kips results in T = 1.382 sec Displacement = 10.31 inch
ARS Force = 1,558 kips Spring Force = 859 kip Spring Stiffness = 804 kip/ft OK within 30% of initial estimate

Seismic Design Parameters

Parameter		GS Section
Performance Criteria	Life Safety – No Collapse	3.2
Earthquake Resisting System	Type 1 – Ductile Substructure with Essentially Elastic Superstructure	3.3
Analysis Procedure	Procedure 2 – Elastic Dynamic Analysis	Table 4.2-1&2
Design Methods for Displacement	Conventional Ductile Response	4.7.1
Local Displacement Capacity	Nonlinear Static Procedure (Pushover)	4.8.2

Seismic Ground Shaking Hazard

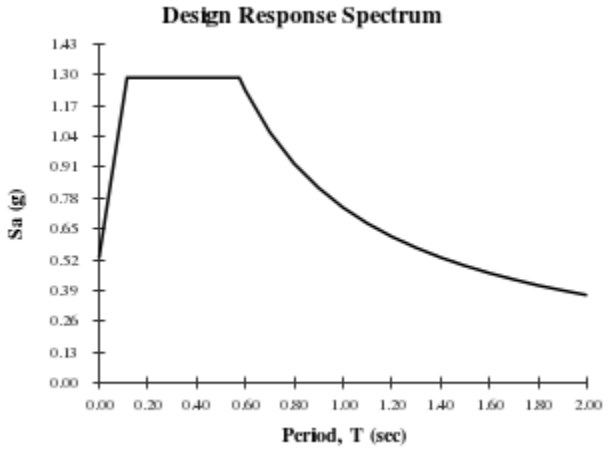
Reference: *GS Section 3.4*

A site-specific hazard analysis is not required as bridge is assumed to be not critical or essential nor does the assumed site consist of Class F soils.

Assumed Parameters from general procedure using USGS/AASHTO Seismic Hazard Maps

Site Class = D

PGA = 0.526g	F_{PGA} = 1.00	A_S = 0.526g
S_S = 1.288g	F_A = 1.00	S_{DS} = 1.288g
S_1 = 0.490g	F_V = 1.510	S_{D1} = 0.740g

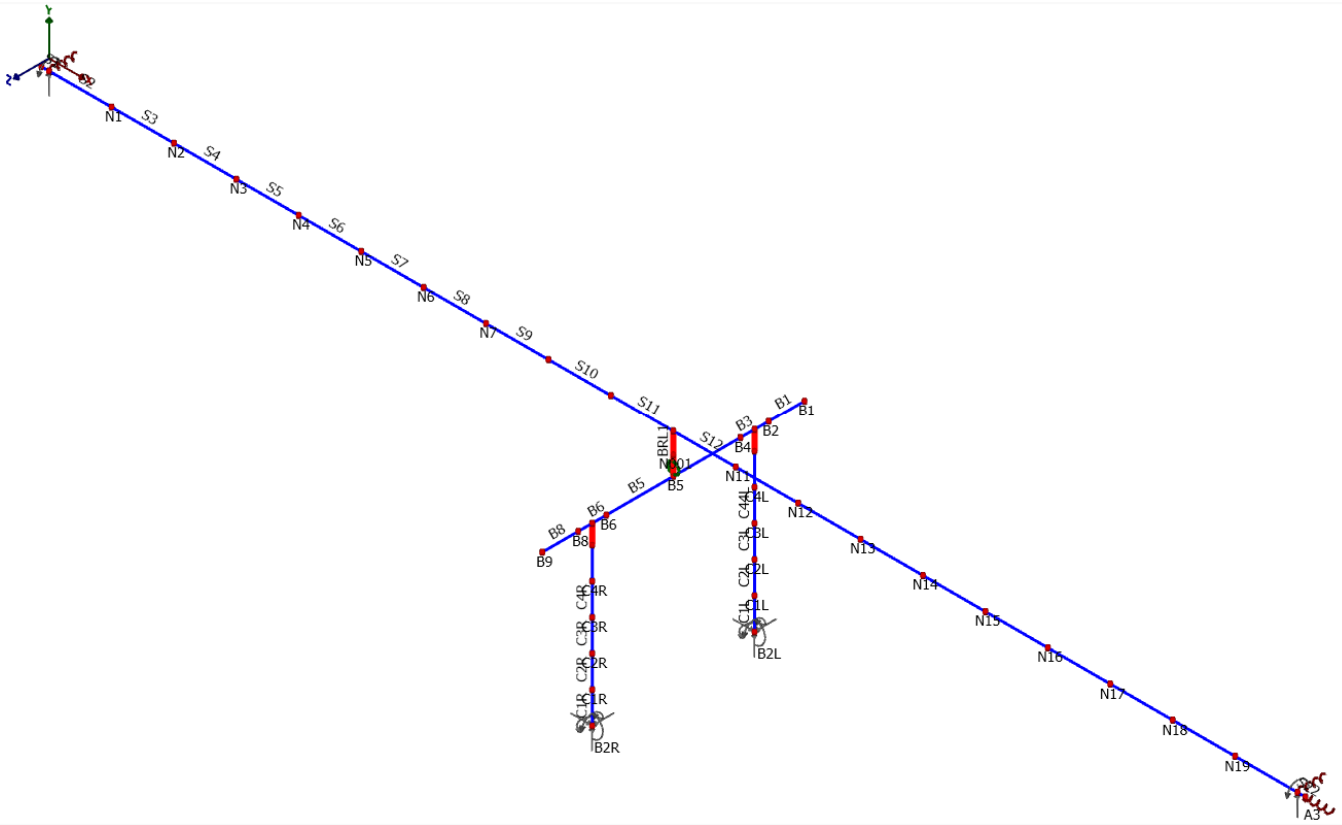


Seismic Design Category = D *GS Table 3.5-1*

Response spectrum applied in in Global X and Z directions using CQC Method and 5% damping

Vertical ground motions are not considered as permitted by GS 4.7.2 as structure is assumed to be not essential or critical

Finite Element Global Model



Modeling Notes

1. Superstructure frame members are located center of gravity located at 1.76 ft from top of deck
2. Superstructure is divided into 10-10.0 ft segments in each span
3. Superstructure includes 1.25 ft cantilever at each abutment
4. Bent 2 cap framing member located at center of gravity located at 8.04 ft from top of deck
5. Rigid link (L= 3.28 ft) connects superstructure to node representing top of bent cap (5.04 ft from top of deck)
6. Rigid link connects node at center of gravity of bent cap to node representing top of bent cap (5.04 ft from top of deck) and this rigid link has Moment Z released at upper end of rigid link to represent the pin connection between superstructure and bent cap
7. Column members are divided into 5-5.0 ft segments
8. Bent 2 column bases are fixed at top of footing (36.04 ft from top of deck) resulting in a total column length of 25 ft
9. Abutments are restrained against vertical displacements and torsion.
10. Z direction (transverse) springs are applied at each abutment
11. X direction (longitudinal) spring is applied at Abutment 3 to represent soil-structure interaction.

Modal Analysis

A modal analysis was performed to obtain 10 mode shapes. Mass includes self weight plus added dead load of barriers and diaphragms.

Modal Results/Participation

Mode Shape Name	frequency (Hz)	Period (Seconds)	X Part	Y Part	Z Part	X Mass K	Y Mass K	Z Mass K
Mode Shape Set 1 Mode 1	0.739	1.353	0.974	0.000	0.000	2748.295	0.000	0.000
Mode Shape Set 1 Mode 2	1.694	0.5905	0.000	0.000	0.983	0.000	0.000	2776.269
Mode Shape Set 1 Mode 3	2.061	0.4852	0.000	0.000	0.000	0.000	0.000	0.000
Mode Shape Set 1 Mode 4	2.962	0.3376	0.000	0.000	0.000	0.119	0.000	0.000
Mode Shape Set 1 Mode 5	4.053	0.2467	0.000	0.703	0.000	0.000	1984.786	0.000
Mode Shape Set 1 Mode 6	9.146	0.1093	0.000	0.029	0.000	0.000	82.486	0.000
Mode Shape Set 1 Mode 7	10.07	0.0993	0.000	0.000	0.000	0.000	0.000	0.020
Mode Shape Set 1 Mode 8	11.75	0.0851	0.000	0.000	0.000	0.728	0.000	0.000
Mode Shape Set 1 Mode 9	15.32	0.0653	0.012	0.000	0.000	32.850	0.000	0.000
Mode Shape Set 1 Mode 10	18.11	0.0552	0.000	0.035	0.000	0.000	97.762	0.000
Mode Shape Set 1 Totals			0.985	0.767	0.983	2781.991	2165.033	2776.289

Mode 1 is longitudinal response
Mode 2 is transverse response

Modal participation > 90% X and Z direction *GS 5.4.3*

Combination of Orthogonal Seismic Displacement Demands

Since this example bridge is not curved and the bents are not skewed, the combination of orthogonal seismic displacement demands is not required per GS 4.4. Orthogonal combinations should be considered with bridges with complex geometry.

Longitudinal Pushover Model

Demand Displacement

Δ_D	7.392	inch	Global Model at top of column

Concrete

w_C	0.145	kcf		
F_{CE}	5.200	ksi		
E_{CE}	4,347	ksi		$E_{CE} = 120{,}000K_1 w_C^{2.0} F_{CE}^{0.33}$

Plastic Hinge

Column D	4.5	feet		
Column L	25	feet	Max Moment to Point Contraflexure	
F_{YE}	68.00	ksi		
Vert Bar Size	# 11			
d_B	1.41	inch		
L_P	38.382	inch	*GS 4.11.6-1*	$L_P = 0.08L + 0.15F_{YE}d_{BL} \geq 0.3F_{YE}d_{BL}$
Region L	6.75	feet	*GS 4.11.7*	

Property	Bottom Column		Source	
Δ_Y	2.910	inch	$\phi_Y L^2/3$	
Axial Load	921	kips	Global Model	
M_P	4,482	ft-kip	Moment Curvature	
I_{CR}	6.454	ft^4	Moment Curvature	
ϕ_Y	0.000097	rad/inch	Moment Curvature	
ϕ_P	0.000858	rad/inch	Moment Curvature	
θ_P	0.032932	radians	$\phi_P L_P$	
Δ_P	9.248	inch	$\theta_P(L-L_P/2)$	
Δ_C	12.158	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.608		*GS 4.8.1*	
Δ_{PD}	4.482		$\Delta_D-\Delta_Y$	*GS 4.9-5*
μ_D	2.540		Δ_D/Δ_Y	
Limit	6.00		*GS 4.9-2*	
V_P	192	kips	$M_P/(L-L_P/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200		*GS 7.3*	
V_{PO}	230	kips	$\gamma_{MO}V_P$	

The distance from the maximum moment at the bottom of the column to the point of zero moment at the top of the cap is actually 31.0 feet, but 25.0 feet is conservatively being used to simplify the computations.

Minimum Lateral Strength

$M_{NE} \geq 0.1P_{TRIB}[(H + 0.5D_S)/\Lambda] = (0.10)(921)(34.28)/1 = 3{,}157$ ft-kip *GS Eqn 8.7.1-1*

$M_{NE} = 4{,}352$ ft-kip

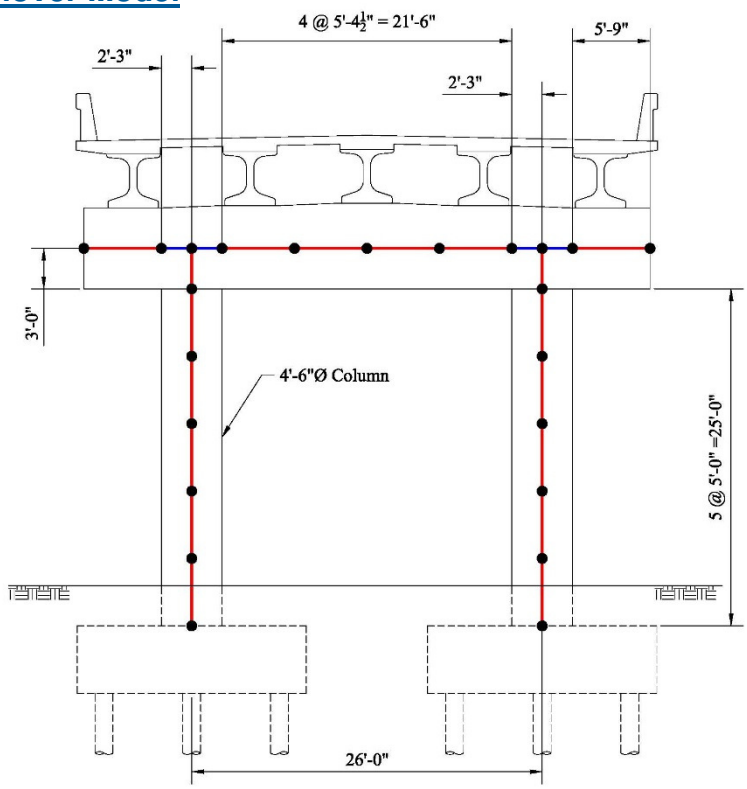
P-Δ Capacity

$P_{DL}\Delta_R \leq 0.25\ M_P$ *GS Eqn 4.11.5-1*

$\Delta_R = 10.31$ inch at top of bent cap

$P_{DL}\Delta_R = (921)(10.31/12) = 792$ ft-kip $< (0.25)(4{,}482) = 1{,}121$ ft-kip

Transverse Pushover Model



PUSHOVER MODEL

A nonlinear static model was created to determine the displacement capacity. The model incrementally pushes the cap laterally to initiate plastic action. As each plastic hinge forms, the model is adjusted to release the moment until all plastic hinges form to create a collapse mechanism.

Member properties and moment capacity increased within joint to reflect increased stiffness and strength

Cap Properties/Loads
DL = (2 column)(921 kip/column reaction at base)/42.0 = 43.86 kip/ft
A = 39.0 ft^2
I_X = 30.24 ft^4
M_{NE} = 8,904 ft-kip

First iteration uses the following column properties based on P = 921 kips
A = 15.092 ft^2
I_X = 6.454 ft^4
M_P = 4,482 ft-kip

The pushover to the right creates an increased compression on the right column and reduced compression on the left column. The column properties were adjusted to account for the revised axial loads as follows:

Location	P	I_X	M_P
Left Column	490 kip	5.692 ft^4	4,001 ft-kip
Right Column	1,352 kip	7.085 ft^4	4,931 ft-kip

Results of revised pushover results in similar axial loads.

Column Type Fixed-Fixed

Demand Displacement

Δ_D	4.240	inch	Global Model at top of column

Concrete

F_C	4.000	ksi

Plastic Hinge

Column D	4.5	feet	
Column L	12.5	feet	Max Moment to Point Contraflexure
F_{YE}	68.00	ksi	
Vert Bar Size	# 11		
d_B	1.41	inch	
L_P	28.764	inch	*GS 4.11.6-1*
Region L_{PR}	6.75	feet	*GS 4.11.7*

Push Model Results Push to Right

	Column Location						
	Left		Right				
Property	Cap	Ftg	Cap	Ftg		Source	
Δ_Y	1.939	1.639	1.601	1.512	inch	Pushover Model	
Axial Load	490	490	1,352	1,352	kips	Pushover Model	
M_P	4,001	4,001	4,931	4,931	ft-kip	Moment Curvature	
I_{CR}	5.69	5.69	7.09	7.09	ft^4	Moment Curvature	
ϕ_P	0.001006	0.001006	0.000733	0.000733	rad/inch	Moment Curvature	
θ_P	0.028937	0.028937	0.021084	0.021084	radians	$\phi_P L_P$	
Δ_P	7.849	7.849	5.719	5.719	inch	$2\theta_P(L-L_P/2)$	
Δ_C	9.787	9.488	7.319	7.231	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.433	0.447	0.579	0.586		*GS 4.8.1*	
Δ_{PD}	2.301	2.601	2.640	2.728		$\Delta_D-\Delta_Y$	
μ_D	2.187	2.587	2.649	2.803		$1+\Delta_{PD}/\Delta_{YI}$	*GS 4.9-5*
Limit	6.00	6.00	6.00	6.00		*GS 4.9-2*	
V_P	354	354	436	436	kips	$M_P/(L-L_P/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200	1.200	1.200	1.200		*GS 7.3*	
V_{PO}	425	425	524	524	kips	$\gamma_{MO}V_P$	
Max P_U	1,832	1,832	1,832	1,832	kips	$0.2F_CA_G$	*GS 8.7.2-1*

NCHRP NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

QUINCY ENGINEERING

Column Shear Capacity

Column Properties

Diameter	54.00	inch	4.500	ft
Area	2,290	in^2	15.904	ft^2
I gross	417,393	in^4	20.129	ft^4
Itorsion	834,786	in^4	40.258	ft^4

Proportions

L	12.50	feet	*Max Moment to Contraflexure*
B_{CAP}	6.50	feet	
Min B_{CAP}	6.50	feet	*GS 8.13.4.1*

Concrete

Aggr Size	1.000	inch	
w_C	0.145	kcf	*AASHTO Table 3.5.1-1*
K_1	1.000		
F_C	4.000	ksi	
E_C	3,987	ksi	*AASHTO 5.4.2.4-1*
F_{CE}	5.200	ksi	*GS 8.4.4-1*
E_{CE}	4,347	ksi	

$E_C = 120{,}000 K_1 w_C^{2.0} F_C^{0.33}$

Vertical Reinforcement

Size	# 11		
d_B	1.41	inch	
Max d_B	2.90	inch	*GS 8.8.6-1*
Area Each Bar	1.56	in^2	
Number	16		
Area	24.96	in^2	
F_{YE}	68.00	ksi	
ρ_v	1.09%		
Min ρ_V	1.00%		*GS Eqn 8.8.2-2*
Max ρ_V	4.00%		*GS Eqn 8.8.1-1*
Bar Circle Radius	23.670	inch	
Bar Spacing	9.295	inch	
Bar Clearance	7.885	inch	
Clear/db	5.59		
Min Clear Space	1.50	inch	*AASHTO 5.10.3.1.1*
L_{AC}	37.87	inch	*GS 8.8.4-1*

Hoop Reinforcement

Clearance	2.00	inch	
Size	# 5		
Diameter	0.625	inch	
Spacing	5.00	inch	
Max Space	8.00	inch	
F_{YH}	60.00	ksi	
Dcore	50.000	inch	*out-out hoop*
A_C	1963.50	in^2	
A_{SP}	0.31	in^2	*GS 8.8.9*
D'	49.38	inch	*center-center hoop*
ρ_S	0.00502		*GS 8.6.2-7*
$\rho_{S\,MIN}$	0.00500		*GS 8.6.5-3*
$\rho_{S\,MIN}$	0.00499		*AASHTO 5.6.4.6-1*
Use $\rho_{S\,MIN}$	0.00500		

Axial Load

	Max	Min		
P_U	1,352	488	kips	
f_A	0.59		ksi	
f_A/F_C	0.15		*Limit = 0.2 GS 8.7.2-1*	*Positive Compression*
				Use Min P_U for Shear

Shear Capacity

A_E	1,832	in^2	*0.8*Agross*	*GS 8.6.2-2*
μ_D	2.80			*GS 4.9-5*
f_S	0.301		$\rho_S F_{YH} \le 0.35$	*GS 8.6.2-6*
α'	2.88		$f_S/0.15+3.67-\mu_D$	*GS 8.6.2-5*
$1+P_U/2A_G$	1.11			
v_C	0.204	ksi		*GS 8.6.2-3*
$v_{C\,MAX}$	0.220	ksi	$0.11\sqrt{f_C}$	
$v_{C\,MAX}$	0.270	ksi	$0.047\alpha'\sqrt{f_C}$	
Use v_C	0.204	ksi		
V_C	373	kips	$v_C A_E$	
V_S	289	kips		*GS 8.6.3-1*
$V_{S\,MAX}$	927	kips	$0.25\sqrt{f_C} A_E$	*GS 8.6.4-1*
$V_{S\,USE}$	289			
V_n	662	kips	V_C+V_S	
ϕ	0.90			*GS 8.6.1-2*
ϕV_n	596	kips		*GS 8.6.1*

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c} \le min\begin{cases} 0.11\sqrt{f_c} \\ 0.047\alpha'\sqrt{f_c} \end{cases}$$

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

$V_{PO} = 524$ kip < 596 kip

Capacity Protected Members

Bent caps and footings shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity M_{PO}. The components shall have adequate shear and moment capacity based upon the expected material properties. The design of these elements is not included.

SEISMIC DESIGN EXAMPLE NO. 2

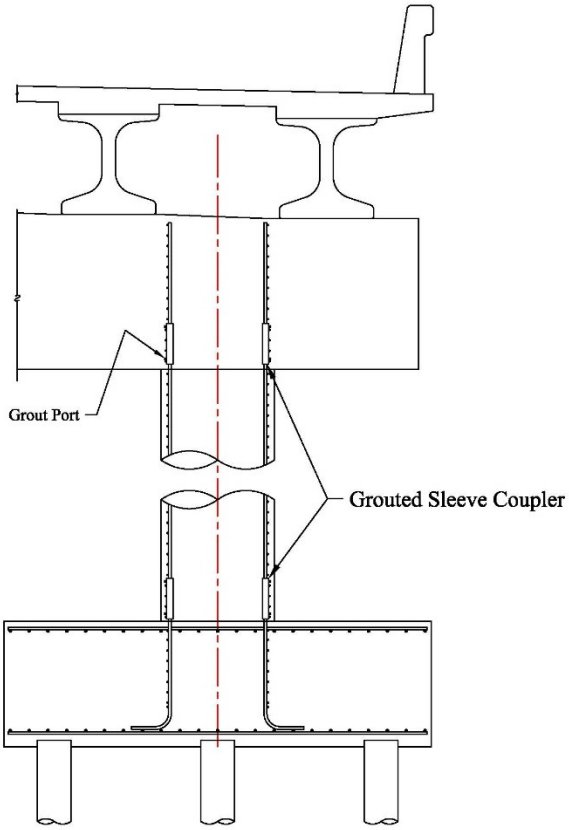
ACCELERATED BRIDGE CONSTRUCTION

COLUMN CONNECTIONS

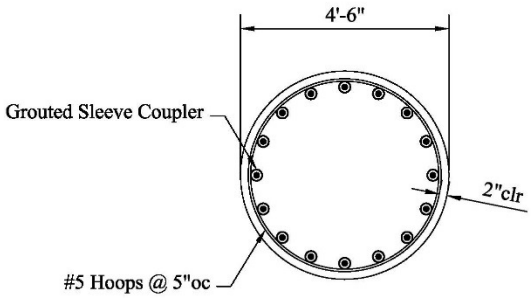
WITH GROUTED SLEEVE COUPLERS

Seismic Design Example No. 2

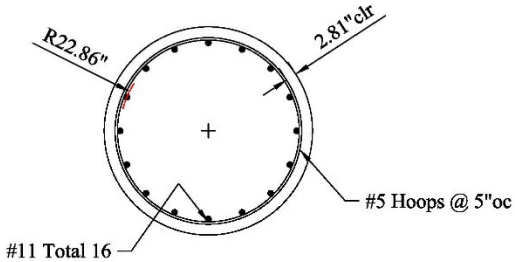
Analyze precast column with grouted sleeve coupler within cap and at bottom of column using bridge described in the conventional bridge example.



Bent Elevation



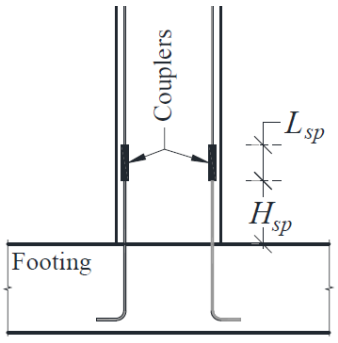
Column at Coupler



Column at Rebar

Coupler Properties

Based upon NMB Splice Sleeve No 11U-X
Coupler Maximum Diameter = 3.03 inch
Coupler Length = L_{SP} = 19.49 inch
Coupler Rigid Length Ratio = β = 0.55 *Table C3.3-1*
H_{SP} = 1.0 inch



Column

Determine revised column properties with vertical reinforcement moved inward to maintain 2.0 inch clear for hoops at couplers. Revised clearance = 2.81 inch above couplers.

Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Column	15.902	20.129	20.129	40.245	2.25	15.902	6.266	6.266	8.05

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
$I_{XEFF} = I_{YEFF} = 6.266 ft^4$ $I_{EFF} = 0.311\ I_G$ for DL = 921 kip

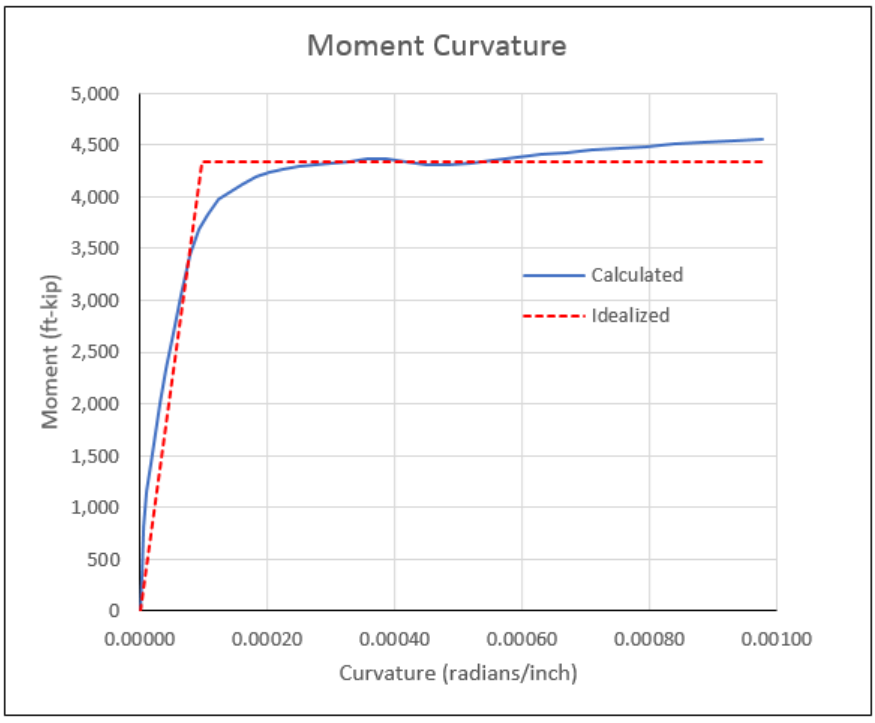
Use16-#11 vertical bars at radius = 22.86 inch ASTM A706 Grade 60 $\rho = 1.09\%$
$f_{CE} = 5.2$ ksi *GS 8.4.4-1*
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi *GS Table 8.4.2-14*
$\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0115$ $\varepsilon^R_{SU} = 0.060$

Use confined concrete within core using Mander's stress-strain model. Use #5 hoops @ 5" oc ASTM A706 Grade 60
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi $\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0150$ $\varepsilon^R_{SU} = 0.090$
Use unconfined concrete outside hoops with $\varepsilon_{CO} = 0.002$ and $\varepsilon_{SP} = 0.005$
Use Axial Load = 921 kip dead load for properties in global model

Effective J = 0.20(40.245) = 8.05 ft^4 *GS 5.6.5-1*

Idealized
$M_P = 4{,}342$ ft-kip
$\phi_Y = 0.000097$ rad/in
$\phi_P = 0.000881$ rad/in
$\phi_U = 0.000978$ rad/in

$M_{NE} = 4{,}280$ ft-kip



Global Model

Global model was updated to reflect revised column properties

Longitudinal Pushover Model

Column Displacement Capacity Column Type Fixed - Pinned

Demand Displacement

Δ_D	7.432	inch	Global Model at top of column with adjusted column stiffness

Concrete

w_C	0.145	kcf	
F_{CE}	5.200	ksi	
E_{CE}	4,347	ksi	$E_{CE} = 120{,}000K_1 w_C^{2.0} F_{CE}^{0.33}$

Plastic Hinge

Column D	4.50	feet		
Column L	25.00	feet	Max Moment to Point Contraflexure	
F_{YE}	68.00	ksi		
Vert Bar Size	# 11			
d_B	1.41	inch		
L_P	38.382	inch	*GS 4.11.6-1*	$L_P = 0.08L + 0.15F_{YE}d_{BL} \geq 0.3F_{YE}d_{BL}$
Region L	6.75	feet	*GS 4.11.7*	

Coupler

L_{SP}	19.49	inch	*Coupler Length*	$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \leq L_p$
H_{SP}	1.00	inch	*Distance column end to coupler face*	
β	0.55		*Coupler Rigid Length Ratio*	
L_P^{SP}	27.942	inch	*Proposed GS Eqn C3.4-1*	

Property	Bottom Column		Source	
Δ_Y	2.896	inch	$\phi_Y L^2/3$	
Axial Load	921	kips	Global Model	
M_P	4,342	ft-kip	Moment Curvature	
I_{CR}	6.266	ft^4	Moment Curvature	
ϕ_Y	0.000097	ft^4	Moment Curvature	
ϕ_P	0.000881	rad/inch	Moment Curvature	
θ_P	0.024617	radians	$\phi_P L_P^{SP}$	
Δ_P	7.041	inch	$\theta_P(L-L_P^{SP}/2)$	
Δ_C	9.937	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.748		*GS 4.8.1*	
Δ_{PD}	4.536		$\Delta_D-\Delta_Y$	*GS 4.9-5*
μ_D	2.567		Δ_D/Δ_Y	
Limit μ_D	6.00		*GS 4.9-2*	
V_P	186	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200		*GS 7.3*	
V_{PO}	223	kips	$\gamma_{MO}V_P$	

Minimum Lateral Strength

$M_{NE} \geq 0.1P_{TRIB}[(H + 0.5D_S)/\Lambda] = (0.10)(921)(34.28)/1 = 3{,}157$ ft-kip *GS Eqn 8.7.1-1*

$M_{NE} = 4{,}280$ ft-kip

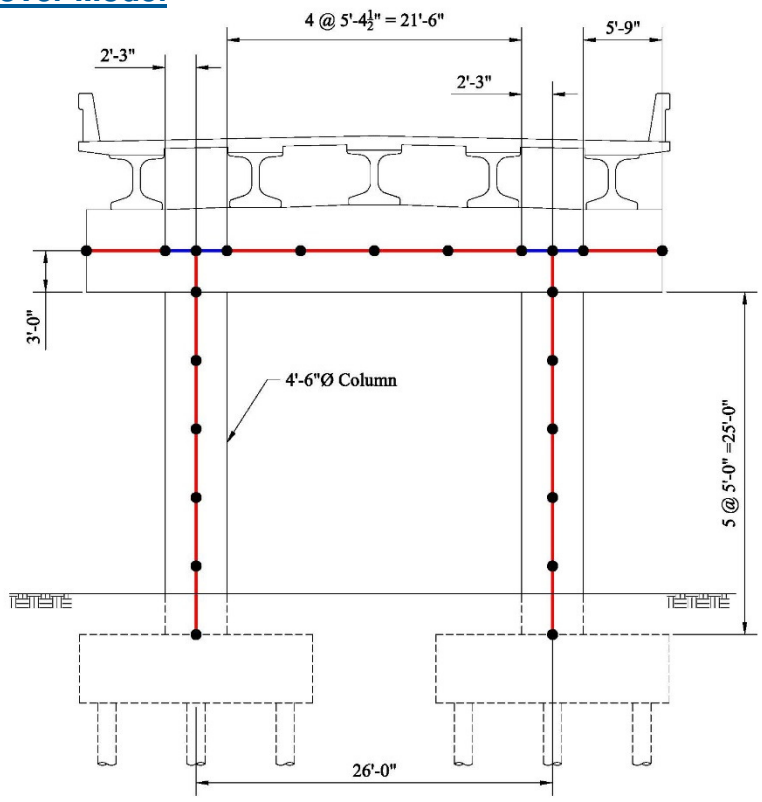
P-Δ Capacity

$P_{DL}\Delta_R \leq 0.25\ M_P$ *GS Eqn 4.11.5-1*

$\Delta_R = 10.36$ inch at top of bent cap

$P_{DL}\Delta_R = (921)(10.36/12) = 795$ ft-kip < $(0.25)(4{,}342) = 1{,}086$ ft-kip

Transverse Pushover Model



PUSHOVER MODEL

A nonlinear static model was created to determine the displacement capacity. The model incrementally pushes the cap laterally to initiate plastic action. As each plastic hinge forms, the model is adjusted to release the moment until all plastic hinges form to create a collapse mechanism.

Member properties and moment capacity increased within joint to reflect increased stiffness and strength

Cap Properties/Loads
DL = (2 column)(921 kip/column reaction at base)/42.0 = 43.86 kip/ft
A = 39.0 ft^2
I_X = 30.24 ft^4
M_{NE} = 8,904 ft-kip

First iteration uses the following column properties based on P = 921 kips
A = 15.092 ft^2
I_X = 6.266 ft^4
M_P = 4,342 ft-kip

The pushover to the right creates an increased compression on the right column and reduced compression on the left column. The column properties were adjusted to account for the revised axial loads as follows:

Location	P	I_X	M_P
Left Column	522 kip	5.558 ft^4	3,913 ft-kip
Right Column	1,319 kip	6.853 ft^4	4,764 ft-kip

Results of revised pushover results in similar axial loads.

Column Type Fixed-Fixed

Demand Displacement

Δ_D	4.273	inch	Global Model at top of column

Concrete

F_C	4.000	ksi

Plastic Hinge

Column D	4.5	feet	
Column L	12.5	feet	Max Moment to Point Contraflexure
F_{YE}	68.00	ksi	
Vert Bar Size	# 11		
d_B	1.41	inch	
L_P	28.764	inch	*GS 4.11.6-1*
Region L_{PR}	6.75	feet	*GS 4.11.7*

Coupler

L_{SP}	19.49	inch	*Coupler Length*
H_{SP}	1.00	inch	*Distance column end to coupler face*
β	0.55		*Coupler Rigid Length Ratio*
L_P^{SP}	18.417	inch	*Proposed GS Eqn C3.4-1*

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \le L_p$$

Push Model Results Push to Right

	Column Location						
	Left		Right				
Property	Cap	Ftg	Cap	Ftg		Source	
Δ_Y	1.933	1.639	1.584	1.509	inch	Pushover Model	
Axial Load	522	522	1,319	1,319	kips	Pushover Model	
M_P	3,913	3,913	4,764	4,764	ft-kip	Moment Curvature	
I_{CR}	5.558	5.558	6.853	6.853	ft^4	Moment Curvature	
ϕ_P	0.001034	0.001034	0.000756	0.000756	rad/inch	Moment Curvature	
θ_P	0.022306	0.019043	0.016309	0.013923	radians	$\phi_P L_P^{SP}$ (Ftg), $0.75\phi_P L_P$ (Cap)	
Δ_P	6.114	5.362	4.471	3.921	inch	$2\theta_P(L-L_P^{SP}/2)$	
Δ_C	8.047	7.001	6.054	5.429	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.531	0.610	0.706	0.787		*GS 4.8.1*	
Δ_{PD}	2.340	2.634	2.689	2.764		$\Delta_D-\Delta_Y$	
μ_D	2.211	2.607	2.698	2.832		$1+\Delta_{PD}/\Delta_{YI}$	*GS 4.9-5*
Limit μ_D	6.00	6.00	6.00	6.00		*GS 4.9-2*	
V_P	343	334	417	406	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200	1.200	1.200	1.200		*GS 7.3*	
V_{PO}	411	400	501	487	kips	$\gamma_{MO}V_P$	
Max P_U	1,832	1,832	1,832	1,832	kips	$0.2F_CA_G$	*GS 8.7.2-1*

Note: 75% L_P used at cap and L_P^{SP} used at footing *Proposed GS C3.4*

Column Shear Capacity

Column Properties

Diameter	54.00	inch	4.500	ft
Area	2,290	in^2	15.904	ft^2
I gross	417,393	in^4	20.129	ft^4
Itorsion	834,786	in^4	40.258	ft^4

Proportions

L	12.50	feet	*Max Moment to Contraflexure*
B_{CAP}	6.50	feet	
Min B_{CAP}	6.50	feet	*GS 8.13.4.1*

Concrete

Aggr Size	1.000	inch	
w_C	0.145	kcf	*AASHTO Table 3.5.1-1*
K_1	1.000		
F_C	4.000	ksi	
E_C	3,987	ksi	*AASHTO 5.4.2.4-1*
F_{CE}	5.200	ksi	*GS 8.4.4-1*
E_{CE}	4,347	ksi	

$E_C = 120{,}000 K_1 w_C^{2.0} F_C^{0.33}$

Vertical Reinforcement

Size	# 11		
d_B	1.41	inch	
Max d_B	2.92	inch	*GS 8.8.6-1*
Area Each Bar	1.56	in^2	
Number	16		
Area	24.96	in^2	
F_{YE}	68.00	ksi	
ρ_V	1.09%		
Min ρ_V	1.00%		*GS Eqn 8.8.2-2*
Max ρ_V	4.00%		*GS Eqn 8.8.1-1*
Bar Circle Radius	22.860	inch	
Bar Spacing	8.977	inch	
Bar Clearance	7.567	inch	
Clear/db	5.37		
Min Clear Space	1.50	inch	*AASHTO 5.10.3.1.1*
L_{AC}	37.87	inch	*GS 8.8.4-1*

Hoop Reinforcement

Clearance	2.81	inch	
Size	# 5		
Diameter	0.625	inch	
Spacing	5.00	inch	
Max Space	8.00	inch	
F_{YH}	60.00	ksi	
Dcore	48.380	inch	*out-out hoop*
A_C	1838.32	in^2	
A_{SP}	0.31	in^2	*GS 8.8.9*
D'	47.76	inch	*center-center hoop*
ρ_S	0.00519		*GS 8.6.2-7*
$\rho_{S\ MIN}$	0.00500		*GS 8.6.5-3*
$\rho_{S\ MIN}$	0.00737		*AASHTO 5.6.4.6-1*
Use $\rho_{S\ MIN}$	0.00737		

#5 Hoops @ 5" oc adequate for shear and ductility, but requires 3.50 inch spacing to conform to AASHTO 5.6.4.6-1

Axial Load

	Max	Min	
P_U	1,319	522	kips
f_A	0.58		ksi
f_A/F_C	0.14		*Limit = 0.2 GS 8.7.2-1*

Positive Compression
Use Min P_U for Shear

Shear Capacity

A_E	1,832	in^2	*0.8*Agross*	*GS 8.6.2-2*
μ_D	2.83			*GS 4.9-5*
f_S	0.312		$\rho_S F_{YH} \le 0.35$	*GS 8.6.2-6*
α'	2.92		$f_S/0.15+3.67-\mu_D$	*GS 8.6.2-5*
$1+P_U/2A_G$	1.11			
v_C	0.208	ksi		*GS 8.6.2-3*
$v_{C\ MAX}$	0.220	ksi	$0.11\sqrt{f_C}$	
$v_{C\ MAX}$	0.274	ksi	$0.047\alpha'\sqrt{f_C}$	
Use v_C	0.208	ksi		
V_C	381	kips	$v_C A_E$	
V_S	279	kips		*GS 8.6.3-1*
$V_{S\ MAX}$	927	kips	$0.25\sqrt{f_C} A_E$	*GS 8.6.4-1*
$V_{S\ USE}$	279			
V_n	660	kips	V_C+V_S	
ϕ	0.90			*GS 8.6.1-2*
ϕV_n	594	kips		*GS 8.6.1*

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c} \le min\begin{Bmatrix}0.11\sqrt{f_c}\\0.047\alpha'\sqrt{f_c}\end{Bmatrix}$$

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

V_{PO} = 501 kip < 594 kip

Capacity Protected Members

Bent caps and footings shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity M_{PO}. The components shall have adequate shear and moment capacity based upon the expected material properties. The design of these elements is not included.

SEISMIC DESIGN EXAMPLE NO. 3

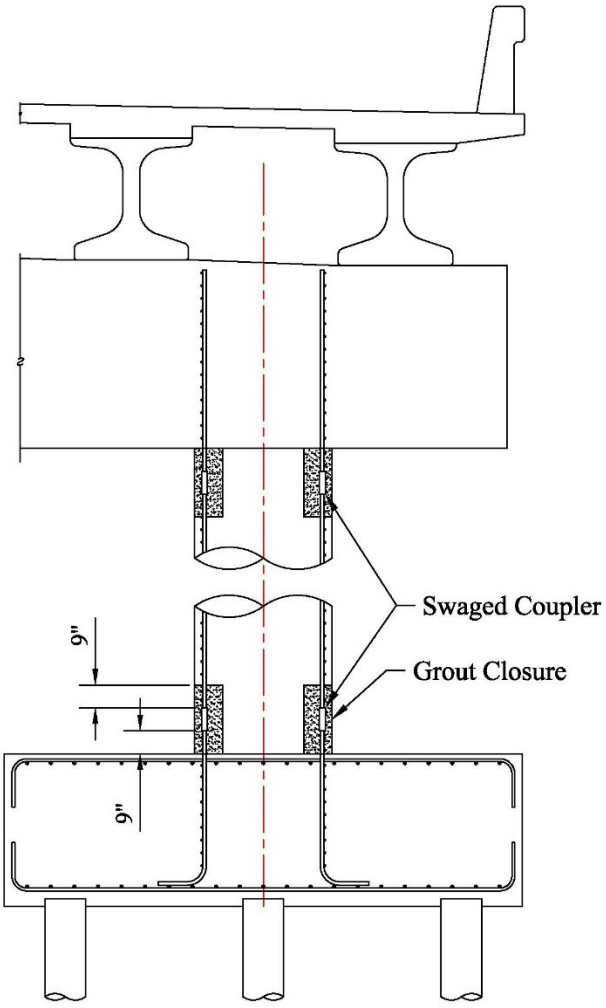
ACCELERATED BRIDGE CONSTRUCTION

COLUMN CONNECTIONS

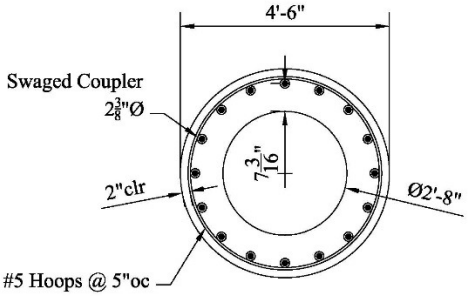
WITH SWAGED COUPLERS

Seismic Design Example No. 3

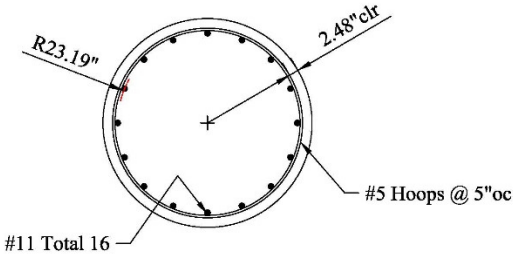
Analyze precast column with swaged sleeve coupler at top and bottom of column using bridge described in the conventional bridge example.



Bent Elevation



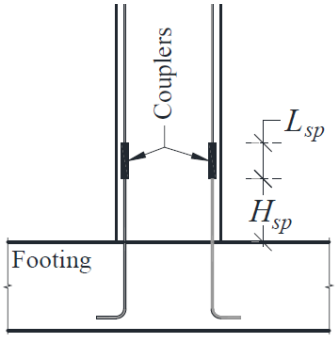
Column at Coupler



Column at Rebar

Coupler Properties
Based upon BarGrip 11XL by BarSplice Products, Inc.
Coupler Maximum Diameter = 2.375 inch
Coupler Length = L_{SP} = 9.0 inch
Coupler Rigid Length Ratio = β = 0.90 *Table C3.3-1*
H_{SP} = 9.0 inch

Clearances above and below the coupler are based upon the manufacturer's clearance recommendations for the swaging equipment.



Column

Determine revised column properties with vertical reinforcement moved inward to maintain 2.0 inch clear for hoops at couplers. Revised clearance = 2.48 inch

Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Column	15.902	20.129	20.129	40.245	2.25	15.902	6.345	6.345	8.05

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
$I_{XEFF} = I_{YEFF} = 6.345\ ft^4$ $I_{EFF} = 0.315\ I_G$ for DL = 921 kip

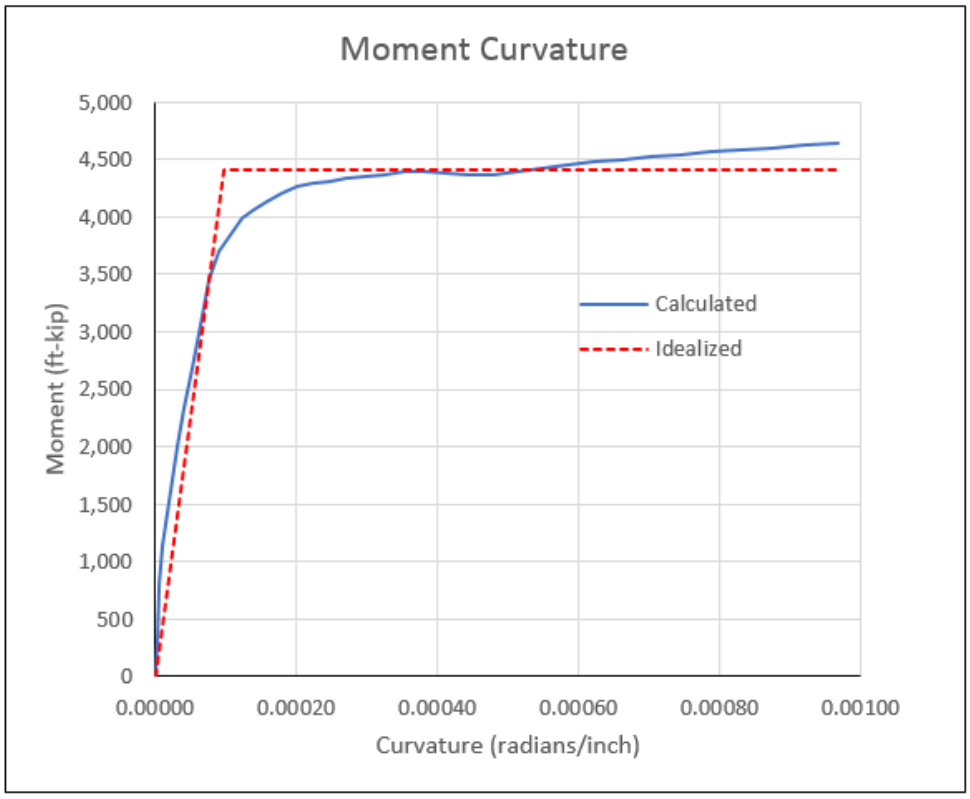
Use16-#11 vertical bars ASTM A706 Grade 60 $\rho = 1.09\%$
$f_{CE} = 5.2$ ksi *GS 8.4.4-1*
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi *GS Table 8.4.2-1*
$\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0115$ $\varepsilon^R_{SU} = 0.060$

Use confined concrete within core using Mander's stress-strain model. Use #5 hoops @ 5" oc ASTM A706 Grade 60
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi $\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0150$ $\varepsilon^R_{SU} = 0.090$
Use unconfined concrete outside hoops with $\varepsilon_{CO} = 0.002$ and $\varepsilon_{SP} = 0.005$
Use Axial Load = 921 kip dead load for properties in global model

Effective J = 0.20(40.245) = $8.05\ ft^4$ *GS 5.6.5-1*

Idealized
$M_P = 4{,}410$ ft-kip
$\phi_Y = 0.000097$ rad/in
$\phi_P = 0.000871$ rad/in
$\phi_U = 0.000968$ rad/in

$M_{NE} = 4{,}310$ ft-kip



Global Model

Global model was updated to reflect revised column properties

Longitudinal Pushover Model

Column Displacement Capacity Column Type Fixed - Pinned

Demand Displacement

Δ_D	7.416	inch	Global Model at top of column with adjusted column stiffness

Concrete

w_C	0.145	kcf	
F_{CE}	5.200	ksi	
E_{CE}	4,347	ksi	$E_{CE} = 120{,}000K_1 w_C^{2.0} F_{CE}^{0.33}$

Plastic Hinge

Column D	4.50	feet		
Column L	25.00	feet	Max Moment to Point Contraflexure	
F_{YE}	68.00	ksi		
Vert Bar Size	# 11			
d_B	1.41	inch		
L_P	38.382	inch	*GS 4.11.6-1*	$L_P = 0.08L + 0.15F_{YE}d_{BL} \geq 0.3F_{YE}d_{BL}$
Region L	6.75	feet	*GS 4.11.7*	

Coupler

L_{SP}	9.00	inch	*Coupler Length*	$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \leq L_p$
H_{SP}	9.00	inch	*Distance column end to coupler face*	
β	0.90		*Coupler Rigid Length Ratio*	
L_P^{SP}	32.181	inch	*Proposed GS Eqn C3.4-1*	

Property	Bottom Column		Source	
Δ_Y	2.904	inch	$\phi_Y L^2/3$	
Axial Load	921	kips	Global Model	
M_P	4,410	ft-kip	Moment Curvature	
I_{CR}	6.345	ft^4	Moment Curvature	
ϕ_Y	0.000097	ft^4	Moment Curvature	
ϕ_P	0.000871	rad/inch	Moment Curvature	
θ_P	0.028030	radians	$\phi_P L_P^{SP}$	
Δ_P	7.958	inch	$\theta_P(L-L_P^{SP}/2)$	
Δ_C	10.862	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.683		*GS 4.8.1*	
Δ_{PD}	4.512		$\Delta_D-\Delta_Y$	*GS 4.9-5*
μ_D	2.554		Δ_D/Δ_Y	
Limit μ_D	6.00		*GS 4.9-2*	
V_P	186	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200		*GS 7.3*	
V_{PO}	224	kips	$\gamma_{MO}V_P$	

Minimum Lateral Strength

$M_{NE} \geq 0.1P_{TRIB}[(H + 0.5D_S)/\Lambda] = (0.10)(921)(34.28)/1 = 3{,}157$ ft-kip *GS Eqn 8.7.1-1*

$M_{NE} = 4{,}310$ ft-kip

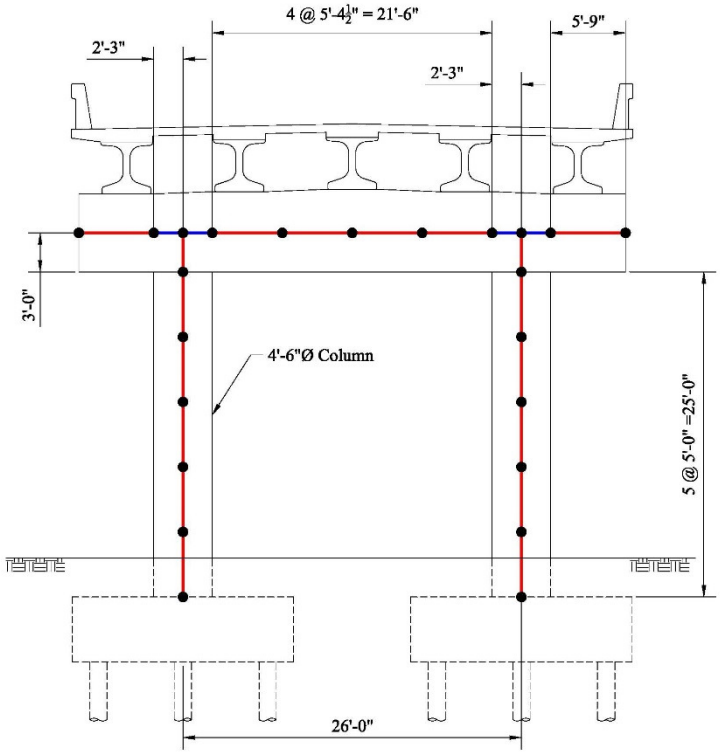
P-Δ Capacity

$P_{DL}\Delta_R \leq 0.25\ M_P$ *GS Eqn 4.11.5-1*

$\Delta_R = 10.338$ inch at top of bent cap

$P_{DL}\Delta_R = (921)(10.338/12) = 793$ ft-kip < $(0.25)(4{,}410) = 1{,}103$ ft-kip

Transverse Pushover Model



PUSHOVER MODEL

A nonlinear static model was created to determine the displacement capacity. The model incrementally pushes the cap laterally to initiate plastic action. As each plastic hinge forms, the model is adjusted to release the moment until all plastic hinges form to create a collapse mechanism.

Member properties and moment capacity increased within joint to reflect increased stiffness and strength

Cap Properties/Loads
DL = (2 column)(921 kip/column reaction at base)/42.0 = 43.86 kip/ft
$A = 39.0\ ft^2$
$I_X = 30.24\ ft^4$
$M_{NE} = 8{,}904$ ft-kip

First iteration uses the following column properties based on P = 921 kips
$A = 15.092\ ft^2$
$I_X = 6.345\ ft^4$
$M_P = 4{,}410$ ft-kip

The pushover to the right creates an increased compression on the right column and reduced compression on the left column. The column properties were adjusted to account for the revised axial loads as follows:

Location	P	I_X	M_P
Left Column	519 kip	$5.608\ ft^4$	3,931 ft-kip
Right Column	1,325 kip	$6.950\ ft^4$	4,832 ft-kip

Results of revised pushover results in similar axial loads.

Column Type Fixed-Fixed

Demand Displacement

Δ_D	4.263	inch	Global Model at top of column

Concrete

F_C	4.000	ksi

Plastic Hinge

Column D	4.5	feet	
Column L	12.5	feet	Max Moment to Point Contraflexure
F_{YE}	68.00	ksi	
Vert Bar Size	# 11		
d_B	1.41	inch	
L_P	28.764	inch	*GS 4.11.6-1*
Region L_{PR}	6.75	feet	*GS 4.11.7*

Coupler

L_{SP}	9.00	inch	*Coupler Length*
H_{SP}	9.00	inch	*Distance column end to coupler face*
β	0.90		*Coupler Rigid Length Ratio*
L_P^{SP}	23.198	inch	*Proposed GS Eqn C3.4-1*

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \le L_p$$

Push Model Results Push to Right

	Column Location						
	Left		Right				
Property	Cap	Ftg	Cap	Ftg		Source	
Δ_Y	1.935	1.635	1.585	1.506	inch	Pushover Model	
Axial Load	519	519	1,325	1,325	kips	Pushover Model	
M_P	3,931	3,931	4,832	4,832	ft-kip	Moment Curvature	
I_{CR}	5.608	5.608	6.950	6.950	ft^4	Moment Curvature	
ϕ_P	0.001023	0.001023	0.000746	0.000746	rad/inch	Moment Curvature	
θ_P	0.023732	0.023732	0.017306	0.017306	radians	$\phi_P L_P^{SP}$	
Δ_P	6.569	6.569	4.790	4.790	inch	$2\theta_P(L-L_P^{SP}/2)$	
Δ_C	8.504	8.204	6.375	6.296	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.501	0.520	0.669	0.677		*GS 4.8.1*	
Δ_{PD}	2.328	2.628	2.678	2.757		$\Delta_D-\Delta_Y$	
μ_D	2.203	2.607	2.690	2.831		$1+\Delta_{PD}/\Delta_{YI}$	*GS 4.9-5*
Limit μ_D	6.00	6.00	6.00	6.00		*GS 4.9-2*	
V_P	341	341	419	419	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200	1.200	1.200	1.200		*GS 7.3*	
V_{PO}	409	409	503	503	kips	$\gamma_{MO}V_P$	
Max P_U	1,832	1,832	1,832	1,832	kips	$0.2F_CA_G$	*GS 8.7.2-1*

Column Shear Capacity

Column Properties

Diameter	54.00	inch	4.500	ft
Area	2,290	in^2	15.904	ft^2
I gross	417,393	in^4	20.129	ft^4
Itorsion	834,786	in^4	40.258	ft^4

Proportions

L	12.50	feet	*Max Moment to Contraflexure*
B_{CAP}	6.50	feet	
Min B_{CAP}	6.50	feet	*GS 8.13.4.1*

Concrete

Aggr Size	1.000	inch	
w_C	0.145	kcf	*AASHTO Table 3.5.1-1*
K_1	1.000		
F_C	4.000	ksi	
E_C	3,987	ksi	*AASHTO 5.4.2.4-1*
F_{CE}	5.200	ksi	*GS 8.4.4-1*
E_{CE}	4,347	ksi	

$E_C = 120{,}000 K_1 w_C^{2.0} F_C^{0.33}$

Vertical Reinforcement

Size	# 11		
d_B	1.41	inch	
Max d_B	2.92	inch	*GS 8.8.6-1*
Area Each Bar	1.56	in^2	
Number	16		
Area	24.96	in^2	
F_{YE}	68.00	ksi	
ρ_V	1.09%		
Min ρ_V	1.00%		*GS Eqn 8.8.2-2*
Max ρ_V	4.00%		*GS Eqn 8.8.1-1*
Bar Circle Radius	23.190	inch	
Bar Spacing	9.107	inch	
Bar Clearance	7.697	inch	
Clear/db	5.46		
Min Clear Space	1.50	inch	*AASHTO 5.10.3.1.1*
L_{AC}	37.87	inch	*GS 8.8.4-1*

Hoop Reinforcement

Clearance	2.48	inch	
Size	# 5		
Diameter	0.625	inch	
Spacing	5.00	inch	
Max Space	8.00	inch	
F_{YH}	60.00	ksi	
Dcore	49.040	inch	*out-out hoop*
A_C	1888.82	in^2	
A_{SP}	0.31	in^2	*GS 8.8.9*
D'	48.42	inch	*center-center hoop*
ρ_S	0.00512		*GS 8.6.2-7*
$\rho_{S\ MIN}$	0.00500		*GS 8.6.5-3*
$\rho_{S\ MIN}$	0.00638		*AASHTO 5.6.4.6-1*
Use $\rho_{S\ MIN}$	0.00638		

#5 Hoops @ 5" oc adequate for shear and ductility, but requires 4.0 inch spacing to conform to AASHTO 5.6.4.6-1

Axial Load

	Max	Min	
P_U	1,325	519	kips
f_A	0.58		ksi
f_A/F_C	0.14		*Limit = 0.2 GS 8.7.2-1*

Positive Compression
Use Min P_U for Shear

Shear Capacity

A_E	1,832	in^2	*0.8*Agross*	*GS 8.6.2-2*
μ_D	2.83			*GS 4.9-5*
f_S	0.307		$\rho_S F_{YH} \le 0.35$	*GS 8.6.2-6*
α'	2.89		$f_S/0.15+3.67-\mu_D$	*GS 8.6.2-5*
$1+P_U/2A_G$	1.11			
v_C	0.206	ksi		*GS 8.6.2-3*
$v_{C\ MAX}$	0.220	ksi	$0.11\sqrt{f_C}$	
$v_{C\ MAX}$	0.271	ksi	$0.047\alpha'\sqrt{f_C}$	
Use v_C	0.206	ksi		
V_C	377	kips	$v_C A_E$	
V_S	283	kips		*GS 8.6.3-1*
$V_{S\ MAX}$	927	kips	$0.25\sqrt{f_C} A_E$	*GS 8.6.4-1*
$V_{S\ USE}$	283			
V_n	660	kips	V_C+V_S	
ϕ	0.90			*GS 8.6.1-2*
ϕV_n	594	kips		*GS 8.6.1*

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c} \le min\begin{Bmatrix} 0.11\sqrt{f_c} \\ 0.047\alpha'\sqrt{f_c} \end{Bmatrix}$$

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

V_{PO} = 503 kip < 594 kip

Capacity Protected Members

Bent caps and footings shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity M_{PO}. The components shall have adequate shear and moment capacity based upon the expected material properties. The design of these elements is not included.

SEISMIC DESIGN EXAMPLE NO. 4

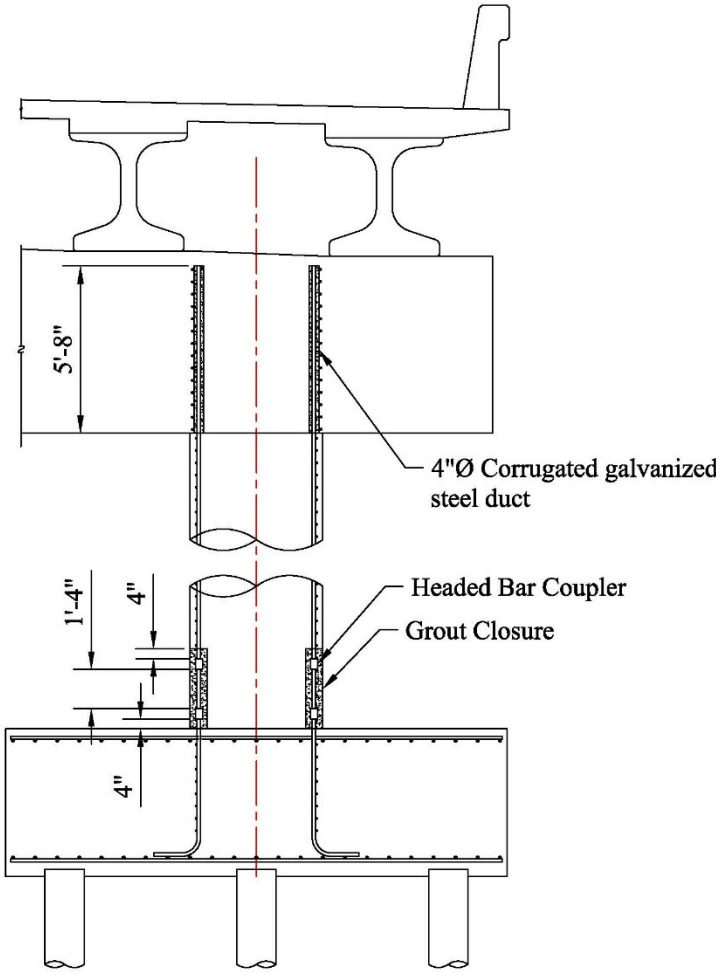
ACCELERATED BRIDGE CONSTRUCTION

COLUMN CONNECTIONS

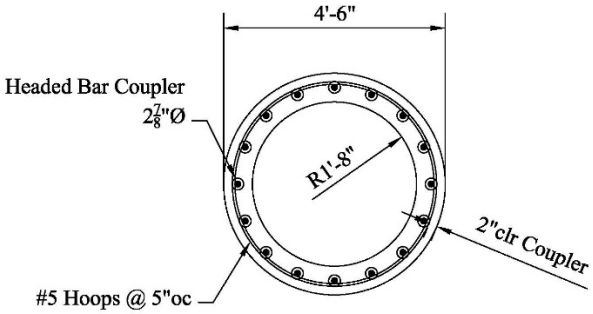
WITH GROUTED DUCTS AND HEADED BAR COUPLERS

Seismic Design Example No. 4

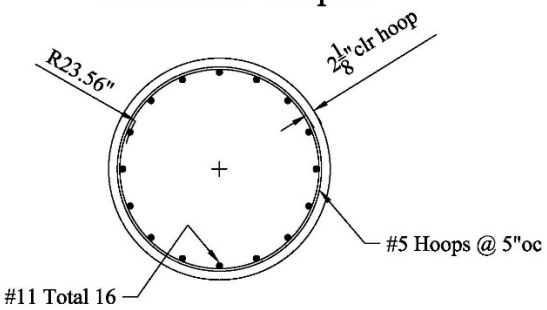
Analyze precast column with headed bar coupler at bottom of column and grouted duct at bent cap using bridge described in the conventional bridge example.



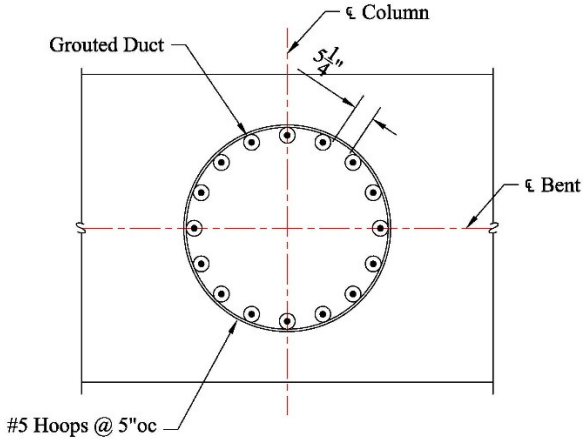
Bent Elevation



Column at Coupler



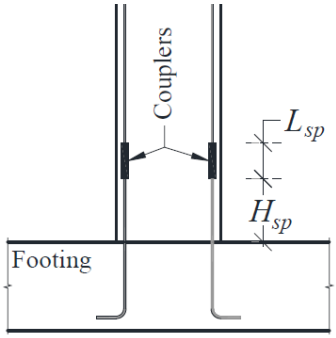
Column at Rebar



Grouted Ducts at Bent Cap

Headed Bar Coupler Properties

Based upon HRC 500/510 Couplers
Coupler Diameter = 2.875 inch
Coupler Length = L_{SP} = 4.25 inch
Coupler Rigid Length Ratio = β = 0.50 *Table C3.3-1*
H_{SP} = 4.0 inch



Grouted Duct Properties

Use 4 inch diameter corrugated galvanized steel ducts which is greater than minimum of 2.75d_B = 3.88 inch
Metal thickness = 0.018 inch (26 ga)
Sheet Metal Specification = ASTM A653
Clear distance between ducts = 5.25 inch > 2.0 inch and 1.3(1.0 inch aggregate) = 1.3 inch
Grout 28 day compressive strength = 7.0 ksi

Grouted Duct Connections *Proposed GS C4.5*

Concrete and Grout

F_C	4.00	ksi
F_G	7.00	ksi

Duct

Diameter d_D	4.00	inch	
Min Dia	3.88	inch	2.75 d_B

Embedment

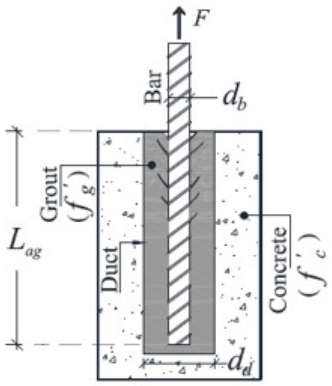
L_{AB}	24.64	inch	*Proposed GS Eqn C4.5-1*
L_{AD}	38.02	inch	*Proposed GS Eqn C4.5-2*
L_{AG}	38.02	inch	

Reinforcement

Size	# 11		
d_{BL}	1.41	inch	
Area	1.56	in^2	
No Bar	1	*per bundle*	
Equiv d_B	1.41	inch	
F_Y	60.00	ksi	
F_{YE}	68.00	ksi	
F_{UE}	95.00	ksi	
F_S	102.00	ksi	Max 1.5F_{YE}, F_{UE}

$$l_{ab} = \frac{0.68 d_{bl} f_{ye}}{\sqrt{f'_g}}$$

$$l_{ad} = \frac{2.25 d^2_{bl} f_{ye}}{d_d \sqrt{f'_c}}$$



Extend grouted duct to far face of bent cap per GS 8.8.4 resulting in embedment length = 68.0 inch > 38.02 inch

Column

Vertical column reinforcement must be relocated to ensure that 2.0 inch clear cover over couplers is maintained. The resulting clear cover to the hoops is revised to 2.125 inch vs 2.00 inch used in the conventional design example. Since the vertical bars are moved only 0.125 inch, it is reasonable to use the column properties used in the conventional design example.

Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Column	15.902	20.129	20.129	40.245	2.25	15.902	6.454	6.454	8.05

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
$I_{XEFF} = I_{YEFF} = 6.454\ ft^4$ $I_{EFF} = 0.321\ I_G$ for DL = 921 kip

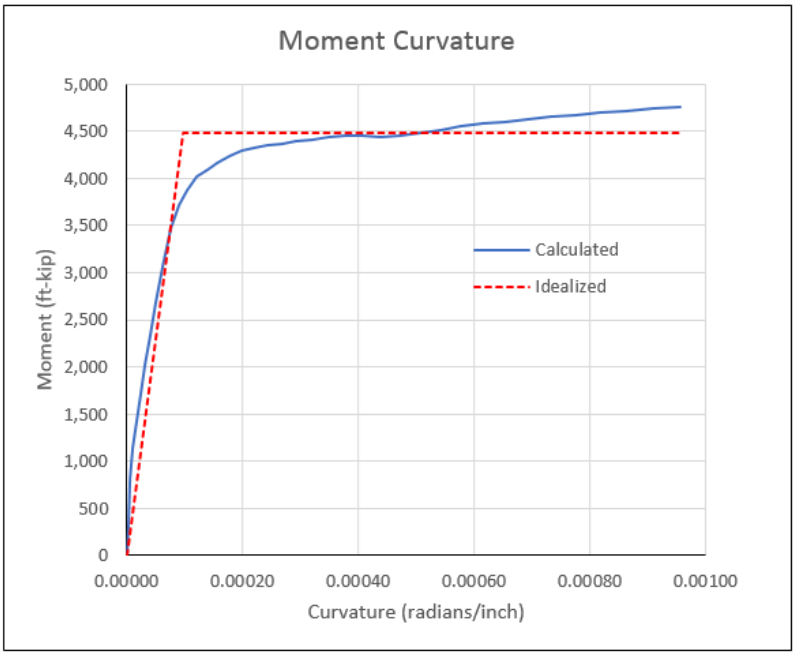
Use16-#11 vertical bars at radius = 23.67 inch ASTM A706 Grade 60 $\rho = 1.09\%$
$f_{CE} = 5.2$ ksi *GS 8.4.4-1*
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi *GS Table 8.4.2-1*
$\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0115$ $\varepsilon^R_{SU} = 0.060$

Use confined concrete within core using Mander's stress-strain model. Use #5 hoops @ 5" oc ASTM A706 Grade 60
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi $\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0150$ $\varepsilon^R_{SU} = 0.090$
Use unconfined concrete outside hoops with $\varepsilon_{CO} = 0.002$ and $\varepsilon_{SP} = 0.005$
Use Axial Load = 921 kip dead load for properties in global model

Effective J = 0.20(40.245) = 8.05 ft^4 *GS 5.6.5-1*

Idealized
$M_P = 4{,}482$ ft-kip
$\phi_Y = 0.000097$ rad/in
$\phi_P = 0.000858$ rad/in
$\phi_U = 0.000955$ rad/in

$M_{NE} = 4{,}352$ ft-kip



Global Model

Utilize member properties and results from conventional design example

Longitudinal Pushover Model

Demand Displacement

Δ_D	7.392	inch	Global Model at top of column

Concrete

w_C	0.145	kcf	
F_{CE}	5.200	ksi	
E_{CE}	4,347	ksi	$E_{CE} = 120{,}000 K_1 w_C^{2.0} F_{CE}^{0.33}$

Plastic Hinge

Column D	4.50	feet		
Column L	25.00	feet	Max Moment to Point Contraflexure	
F_{YE}	68.00	ksi		
Vert Bar Size	# 11			
d_B	1.41	inch		
L_P	38.382	inch	*GS 4.11.6-1*	$L_P = 0.08L + 0.15 F_{YE} d_{BL} \geq 0.3 F_{YE} d_{BL}$
Region L	6.75	feet	*GS 4.11.7*	

Coupler

L_{SP}	4.25	inch	*Coupler Length*
H_{SP}	4.00	inch	*Distance column end to coupler face*
β	0.50		*Coupler Rigid Length Ratio*
L_P^{SP}	36.478	inch	*Proposed GS Eqn C3.4-1*

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \leq L_p$$

Property	Bottom Column		Source	
Δ_Y	2.910	inch	$\phi_Y L^2/3$	
Axial Load	921	kips	Global Model	
M_P	4,482	ft-kip	Moment Curvature	
I_{CR}	6.454	ft^4	Moment Curvature	
ϕ_Y	0.000097	ft^4	Moment Curvature	
ϕ_P	0.000858	rad/inch	Moment Curvature	
θ_P	0.031299	radians	$\phi_P L_P^{SP}$	
Δ_P	8.819	inch	$\theta_P(L-L_P^{SP}/2)$	
Δ_C	11.729	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.630		*GS 4.8.1*	
Δ_{PD}	4.482		$\Delta_D-\Delta_Y$	*GS 4.9-5*
μ_D	2.540		Δ_D/Δ_Y	
Limit μ_D	6.00		*GS 4.9-2*	
V_P	192	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200		*GS 7.3*	
V_{PO}	230	kips	$\gamma_{MO} V_P$	

Minimum Lateral Strength

$M_{NE} \geq 0.1 P_{TRIB}[(H + 0.5D_S)/\Lambda] = (0.10)(921)(34.28)/1 = 3{,}157$ ft-kip *GS Eqn 8.7.1-1*

$M_{NE} = 4{,}352$ ft-kip

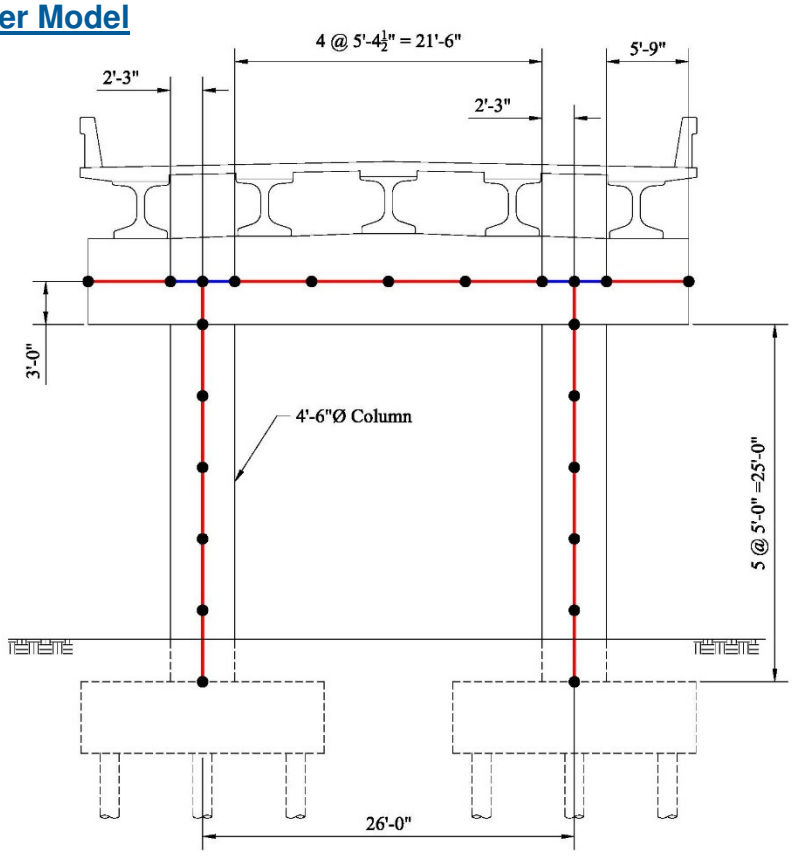
P-Δ Capacity

$P_{DL}\Delta_R \leq 0.25\ M_P$ *GS Eqn 4.11.5-1*

$\Delta_R = 10.31$ inch at top of bent cap

$P_{DL}\Delta_R = (921)(10.31/12) = 792$ ft-kip < $(0.25)(4{,}482) = 1{,}121$ ft-kip

Transverse Pushover Model



PUSHOVER MODEL

A nonlinear static model was created to determine the displacement capacity. The model incrementally pushes the cap laterally to initiate plastic action. As each plastic hinge forms, the model is adjusted to release the moment until all plastic hinges form to create a collapse mechanism.

Member properties and moment capacity increased within joint to reflect increased stiffness and strength

Cap Properties/Loads
DL = (2 column)(921 kip/column reaction at base)/42.0 = 43.86 kip/ft
$A = 39.0\ ft^2$
$I_X = 30.24\ ft^4$
$M_{NE} = 8{,}904$ ft-kip

First iteration uses the following column properties based on P = 921 kips
$A = 15.092\ ft^2$
$I_X = 6.454\ ft^4$
$M_P = 4{,}482$ ft-kip

The pushover to the right creates an increased compression on the right column and reduced compression on the left column. The column properties were adjusted to account for the revised axial loads as follows:

Location	P	I_X	M_P
Left Column	490 kip	5.692 ft^4	4,001 ft-kip
Right Column	1,352 kip	7.085 ft^4	4,931 ft-kip

Results of revised pushover results in similar axial loads.

Column Type Fixed-Fixed

Demand Displacement

Δ_D	4.240	inch	Global Model at top of column

Concrete

F_C	4.000	ksi

Plastic Hinge

Column D	4.5	feet	
Column L	12.5	feet	Max Moment to Point Contraflexure
F_{YE}	68.00	ksi	
Vert Bar Size	# 11		
d_B	1.41	inch	
L_P	28.764	inch	*GS 4.11.6-1*
Region L_{PR}	6.75	feet	*GS 4.11.7*

Coupler

L_{SP}	4.25	inch	*Coupler Length*
H_{SP}	4.00	inch	*Distance column end to coupler face*
β	0.50		*Coupler Rigid Length Ratio*
L_P^{SP}	26.935	inch	*Proposed GS Eqn C3.4-1*

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \le L_p$$

Push Model Results Push to Right

	Column Location						
	Left		Right				
Property	Cap	Ftg	Cap	Ftg		Source	
Δ_Y	1.939	1.639	1.601	1.512	inch	Pushover Model	
Axial Load	490	490	1,352	1,352	kips	Pushover Model	
M_P	4,001	4,001	4,931	4,931	ft-kip	Moment Curvature	
I_{CR}	5.69	5.69	7.09	7.09	ft^4	Moment Curvature	
ϕ_P	0.001006	0.001006	0.000733	0.000733	rad/inch	Moment Curvature	
θ_P	0.028937	0.027096	0.021084	0.019743	radians	$\phi_P L_P^{SP}$	
Δ_P	7.849	7.399	5.719	5.391	inch	$2\theta_P(L-L_P^{SP}/2)$	
Δ_C	9.787	9.038	7.319	6.904	inch	$\Delta_Y+\Delta_P$	
Δ_D/Δ_C	0.433	0.469	0.579	0.614		*GS 4.8.1*	
Δ_{PD}	2.301	2.601	2.640	2.728		$\Delta_D-\Delta_Y$	
μ_D	2.187	2.587	2.649	2.803		$1+\Delta_{PD}/\Delta_{YI}$	*GS 4.9-5*
Limit μ_D	6.00	6.00	6.00	6.00		*GS 4.9-2*	
V_P	354	352	436	433	kips	$M_P/(L-L_P^{SP}/2)$	*GS Figure 4.11.2-3*
γ_{MO}	1.200	1.200	1.200	1.200		*GS 7.3*	
V_{PO}	425	422	524	520	kips	$\gamma_{MO}V_P$	
Max P_U	1,832	1,832	1,832	1,832	kips	$0.2F_CA_G$	*GS 8.7.2-1*

Notes:

1. L_P used at cap and L_P^{SP} used at footing

Column Shear Capacity

Column Properties

Diameter	54.00	inch	4.500	ft
Area	2,290	in^2	15.904	ft^2
I gross	417,393	in^4	20.129	ft^4
Itorsion	834,786	in^4	40.258	ft^4

Proportions

L	12.50	feet	*Max Moment to Contraflexure*
B_{CAP}	6.50	feet	
Min B_{CAP}	6.50	feet	*GS 8.13.4.1*

Vertical Reinforcement

Size	# 11		
d_B	1.41	inch	
Max d_B	2.90	inch	*GS 8.8.6-1*
Area Each Bar	1.56	in^2	
Number	16		
Area	24.96	in^2	
F_{YE}	68.00	ksi	
ρ_v	1.09%		
Min ρ_V	1.00%	*GS Eqn 8.8.2-2*	
Max ρ_V	4.00%	*GS Eqn 8.8.1-1*	
Bar Circle Radius	23.670	inch	
Bar Spacing	9.295	inch	
Bar Clearance	7.885	inch	
Clear/db	5.59		
Min Clear Space	1.50	inch	*AASHTO 5.10.3.1.1*
L_{AC}	37.87	inch	*GS 8.8.4-1*

Axial Load

	Max	Min		
P_U	1,352	490	kips	
f_A	0.59		ksi	
f_A/F_C	0.15		*Limit = 0.2 GS 8.7.2-1*	*Positive Compression*
				Use Min P_U for Shear

Shear Capacity

A_E	1,832	in^2	*0.8*Agross*	*GS 8.6.2-2*
μ_D	2.80			*GS 4.9-5*
f_S	0.301		$\rho_S F_{YH} \leq 0.35$	*GS 8.6.2-6*
α'	2.88		$f_S/0.15+3.67-\mu_D$	*GS 8.6.2-5*
$1+P_U/2A_G$	1.11			
v_C	0.204	ksi		*GS 8.6.2-3*
$v_{C\ MAX}$	0.220	ksi	$0.11\sqrt{f_C}$	
$v_{C\ MAX}$	0.270	ksi	$0.047\alpha'\sqrt{f_C}$	
Use v_C	0.204	ksi		
V_C	373	kips	$v_C A_E$	
V_S	289	kips		*GS 8.6.3-1*
$V_{S\ MAX}$	927	kips	$0.25\sqrt{f_C}A_E$	*GS 8.6.4-1*
$V_{S\ USE}$	289			
V_n	662	kips	V_C+V_S	
ϕ	0.90			*GS 8.6.1-2*
ϕV_n	596	kips		*GS 8.6.1*

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c} \leq min\begin{Bmatrix} 0.11\sqrt{f_c} \\ 0.047\alpha'\sqrt{f_c} \end{Bmatrix}$$

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

V_{PO} = 524 kip < 596 kip

Capacity Protected Members

Bent caps and footings shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity M_{PO}. The components shall have adequate shear and moment capacity based upon the expected material properties. The design of these elements is not included.

SEISMIC DESIGN EXAMPLE NO. 5

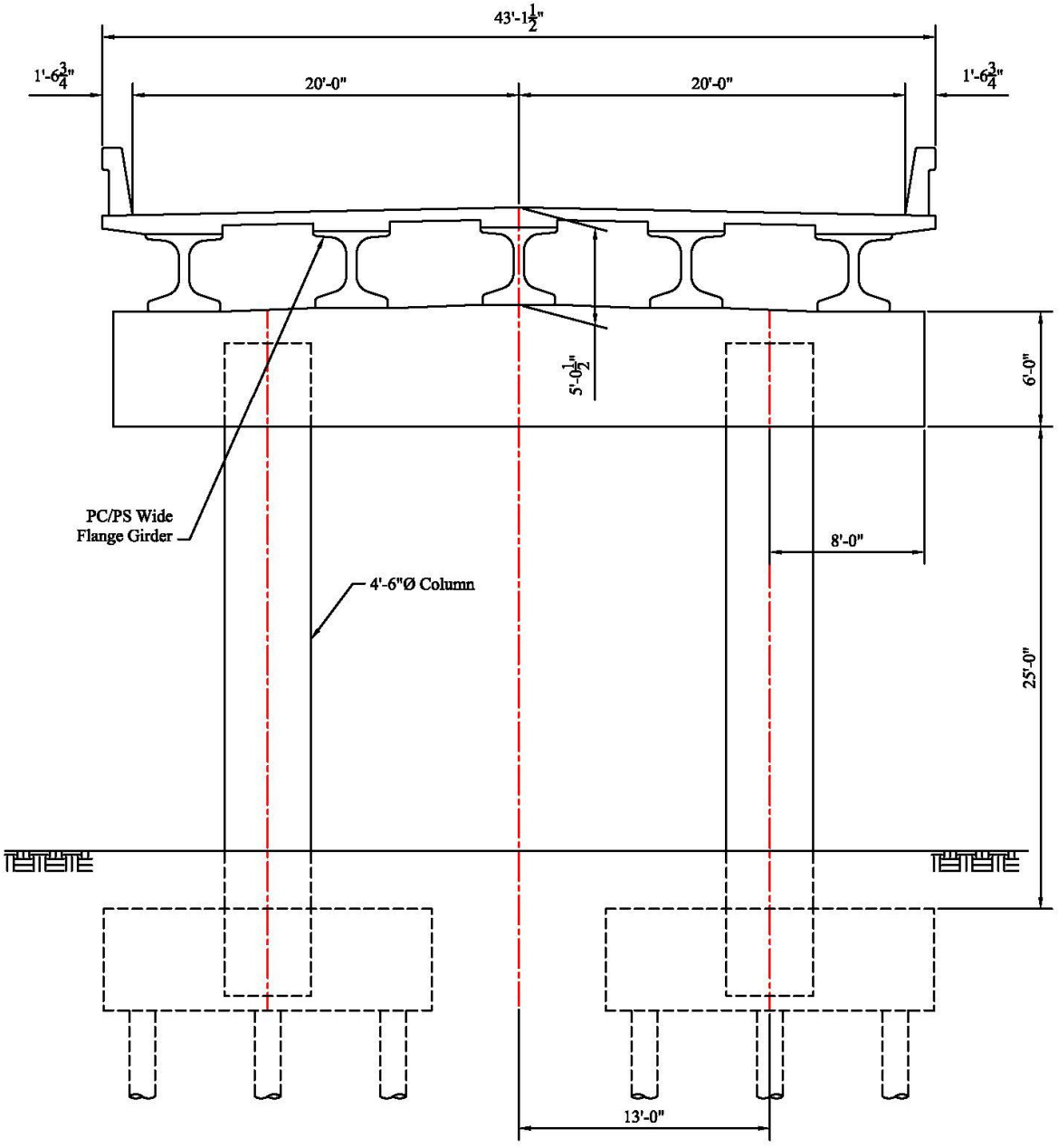
ACCELERATED BRIDGE CONSTRUCTION

COLUMN CONNECTIONS

WITH POCKET CONNECTIONS

Seismic Design Example No. 5

Analyze precast column with pocket connection at top and bottom of column using bridge described in the conventional bridge example.

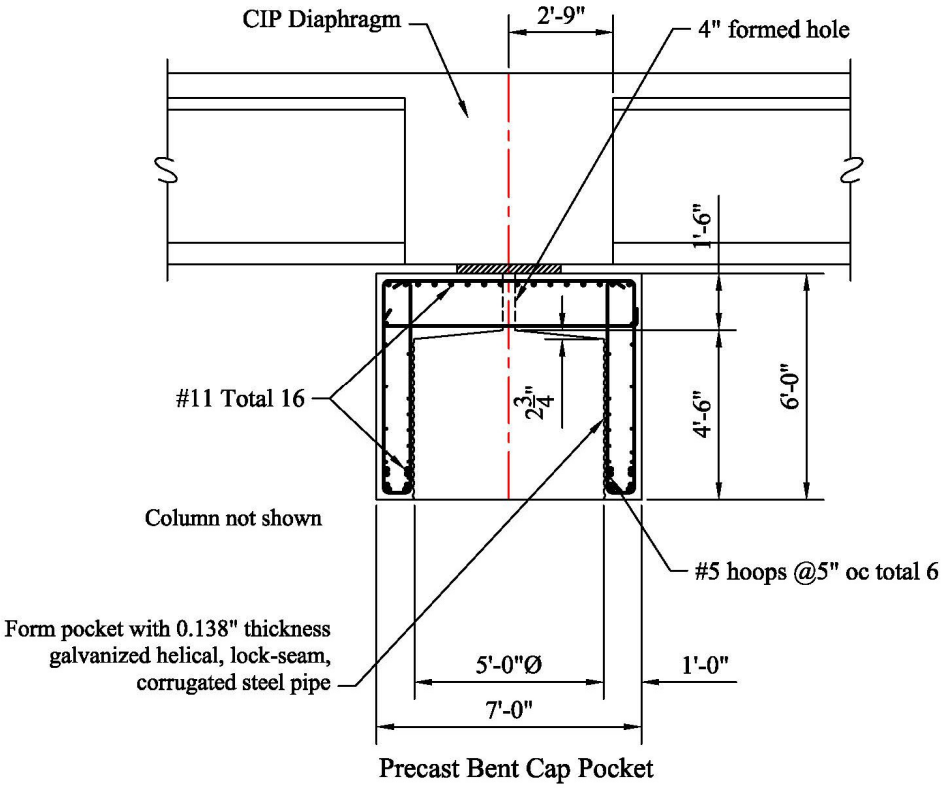


TYPICAL SECTION

References:

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011 with 2012, 2014 and 2015 interims (GS)
2. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (AASHTO LRFD)

Bent Cap



Precast Bent Cap Pocket

Element	Gross Properties				
	Area	I_X	I_Y	J	Y_C
	ft^2	ft^4	ft^4	ft^4	feet
Bent Cap	42.0	126.0	171.5	244.8	3.0

Element	Effective Properties				
	Area	I_X	I_Y	J	M_P
	ft^2	ft^4	ft^4	ft^4	ft-kip
Bent Cap Pos	42.0	27.502	1715	48.96	10,518
Bent Cap Neg	42.0	30.097	1715	48.96	10,786

Determine effective moment of inertia and plastic moment using moment curvature analysis and unconfined concrete

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*

Use $I_X = 28.80\ ft^4$ average

Use16-#11 top and bottom

$f_{CE} = 5.2$ ksi *GS Eqn 8.4.4-1*

$F_{YE} = 68.0$ ksi *GS Table 8.4.2-14*

$\varepsilon_{CU} = 0.005$

Use 10x I_Y as bent cap is restrained by superstructure

Effective $I_Y = 10(171.5) = 1715\ ft^4$

Effective $J = 0.20(244.8) = 48.96\ ft^4$ *GS 5.6.5-1*

Beta = 90 deg

Pocket Connection Requirements at Cap

Minimum Cap Width = D_{CM} + 2.5 = 7.0 ft — *Proposed GS C4.2.2*
Pocket Depth = $H_P \geq 1.0D_{CM}$ = 1.0(4.50) = 4.50 ft — *Proposed GS Figure C4.2.1-2*
$H_P \geq 0.79d_{BL}F_{YE}/\sqrt{F_C}$ = 0.79(1.41)(68)/$\sqrt{4.0}$ = 37.9 inch = 3.16 ft — *Proposed GS Eqn C4.1.1-2*
$H_P \geq 24d_{BL}$ = 24(1.41) = 33.84 inch = 2.82 ft — *Proposed GS Eqn C4.1.1-3*

Minimum Depth above Socket = $0.25H_P$ = 0.25(4.50) = 1.125 ft — *Proposed GS Figure C4.2.1-2*
Use 1.50 ft
Use H_{CAP} = 4.50 + 1.50 = 6.0 ft = 1.33D_C

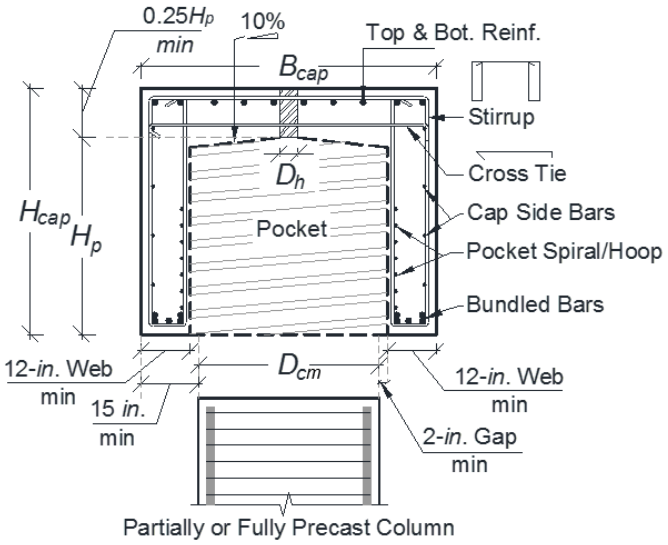
Place transverse reinforcement in the lower half of the bent cap using the same reinforcement as the column

Pocket Material
Pockets shall be constructed with helical, lock-seam, corrugated steel pipes conforming to ASTM A760.
Diameter = 60 inch

$t_p = A_{sp}.f_{yh}/(S_h.f_{yp}.\cos\theta) \geq 0.06$ in. — *Eqn C4.1.2-1*

t_P = (0.31)(60)/[(5.0)(30)cos(20)] = 0.131 inch

Use 0.138 inch thickness



Cap Dead Load
DL = (42.0)(6.0)(7.0)(0.150 kcf) – (19.6 pocket area)(2 columns)(4.50 pocket)(0.150 kcf) =
DL = 264.6 – 26.5 = 238.1 kip

Punching Shear
V_O = (1.25 Load Factor)(238.1)/2 column = 149 kip
β_C = 1.0
h = 20.75 inch
d_E = 17.0 – 0.75/2 = ≈ 16.5 inch
d_V = 0.90(16.5) = 14.85 inch — *AASHTO LRFD 5.7.2.8*
d_V = 0.72(20.75) = 14.94 inch
b_O = π(60+14.94) = 235 inch
λ = 1.0 — *AASHTO LRFD 5.4.2.8*
V_N = (0.063 + 0.126/1.0)(1.0)$\sqrt{4.0}$(235)(14.94) = 1320 kip
V_N = (0.126)(1.0)$\sqrt{4.0}$(235)(14.94) = 885 kip

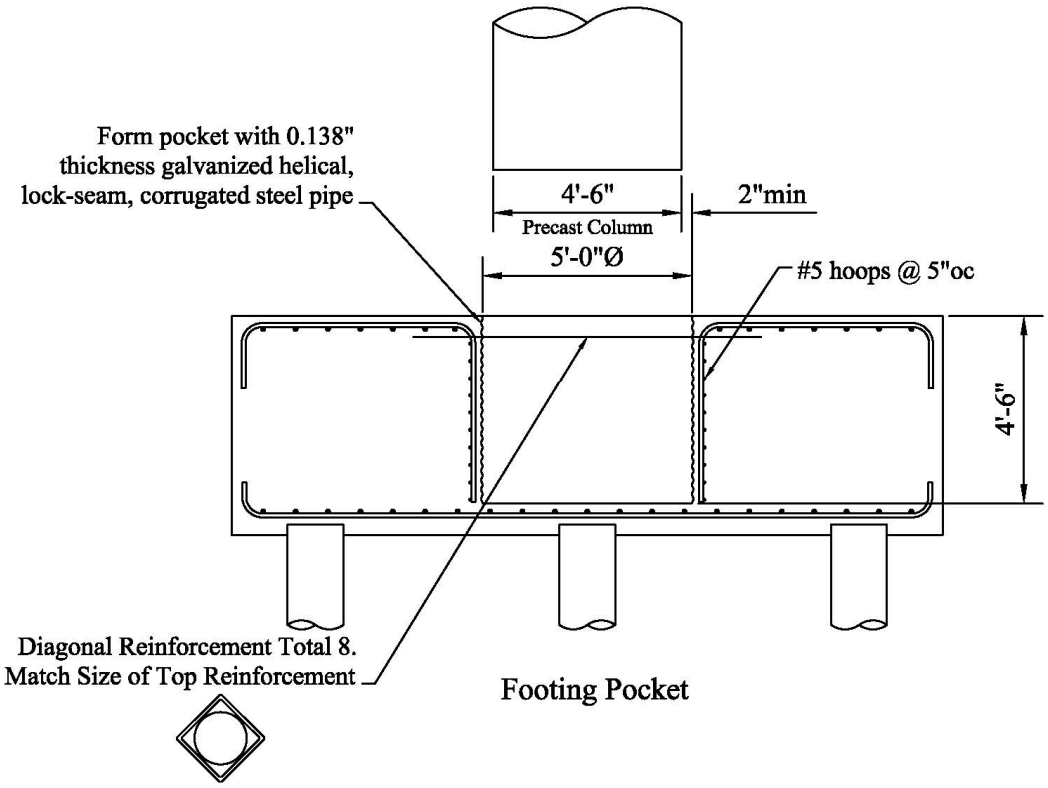
$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\lambda\sqrt{f_c'}\,b_o d_v \leq 0.126\,\lambda\sqrt{f_c'}\,b_o d_v \qquad (5.12.8.6.3\text{-}1)$$

ϕ = 0.90 — *AASHTO LRFD 5.5.4.2*
ϕV_N = 0.90(885) = 796 kip

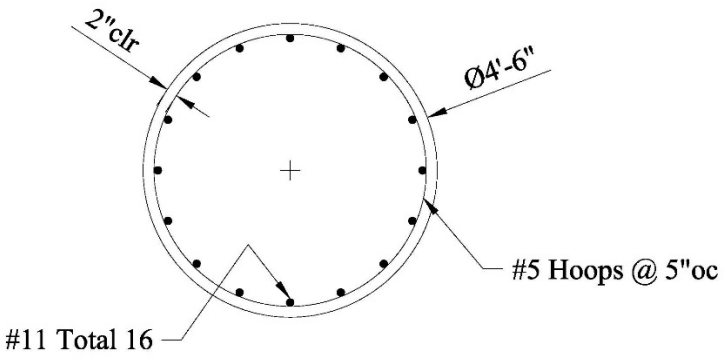
Check ACI 318-14 Section 22.6.5.2 Eqn (c)
v_C = [2.0+(20)(14.94)/235](1.0)$\sqrt{4000}$ = 207 psi
ϕV_N = (0.75)(207)(235)(14.94)/1000 = 545 kip > 149 kip

Pocket Connection Requirements at Footing

Use same pocket dimensions as cap



Column



Element	Gross Properties					Effective Properties			
	Area	I_X	I_Y	J	Y_C	Area	I_X	I_Y	J
	ft^2	ft^4	ft^4	ft^4	feet	ft^2	ft^4	ft^4	ft^4
Column	15.902	20.129	20.129	40.245	2.25	15.902	6.454	6.454	8.05

Effective I_X based on initial slope of M/ϕ *GS Eqn 5.6.2-1*
$I_{XEFF} = I_{YEFF} = 6.454\ ft^4$ $I_{EFF} = 0.321\ I_G$ for DL = 921 kip

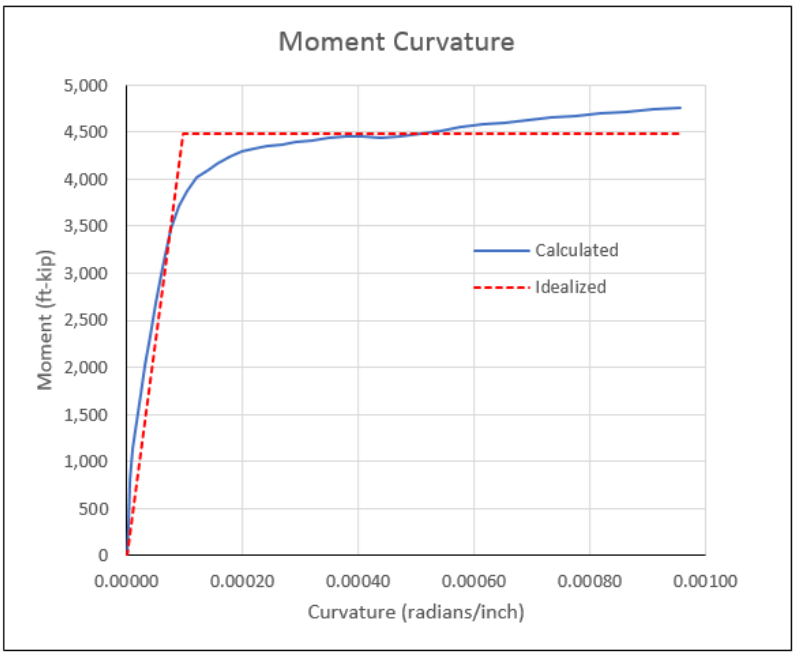
Use16-#11 vertical bars at radius = 23.67 inch ASTM A706 Grade 60 ρ = 1.09%
$f_{CE} = 5.2$ ksi *GS 8.4.4-1*
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi *GS Table 8.4.2-1*
$\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0115$ $\varepsilon^R_{SU} = 0.060$

Use confined concrete within core using Mander's stress-strain model. Use #5 hoops @ 5" oc ASTM A706 Grade 60
$f_{YE} = 68.0$ ksi $f_{UE} = 95.0$ ksi $\varepsilon_{YE} = 0.0023$ $\varepsilon_{SH} = 0.0150$ $\varepsilon^R_{SU} = 0.090$
Use unconfined concrete outside hoops with $\varepsilon_{CO} = 0.002$ and $\varepsilon_{SP} = 0.005$
Use Axial Load = 921 kip dead load for properties in global model

Effective J = 0.20(40.245) = $8.05\ ft^4$ *GS 5.6.5-1*

Idealized
$M_P = 4{,}482$ ft-kip
$\phi_Y = 0.000097$ rad/in
$\phi_P = 0.000858$ rad/in
$\phi_U = 0.000955$ rad/in

$M_{NE} = 4{,}352$ ft-kip

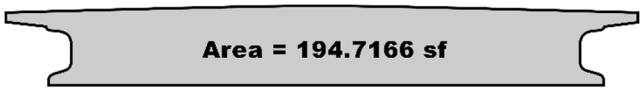


Dead Loads

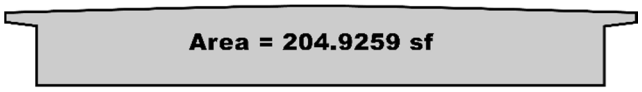
Element	Load Case	No	Area ft^2	w_{DL} klf	Length Feet	DL kip	Comments
Superstructure	SW	1	66.71	10.00	202.5	2026	
Bent Cap	SW	1	42.00	6.30	42.0	265	
Column	SW	2	15.902	2.385	25.00	119	
Total Self Wt						2410	
Barriers	DC	2	3.715	1.11	202.5	225	
Abutment Diaphragm	DC	2	138.2	20.73	2.5	104	
Bent Cap Diaphragm	DC	1	128.0	19.20	5.5	106	
Total Diaphragm						210	
Added DL						434	
Total DL						2844	

Assume no wearing surface and no utilities
Net Area Bent Cap Diaphragm = 194.72-66.71 = 128.0 ft^2
Net Area Abutment Diaphragm = 204.93-66.71 = 138.2 ft^2

SW = Self Weight
DC = DL components/attachments



Bent Cap Diaphragm



Abutment Diaphragm

Statics Check Information

Result Case Name	Applied DY K
Added DL	-434
Barrier	-225
Diaph	-209
Self Wt	-2410
Total DL	-2844
Unit Load	0

Live Loads
Not considered

Load Combinations
Extreme Event I *AASHTO LRFD Table 3.4.1-1*

Load and Resistance Factors
$\phi = 1.0$ $\gamma_{EQ} = 0.0$ $\gamma_D = 1.0$ *GS 3.70*

Bent Foundation Stiffness

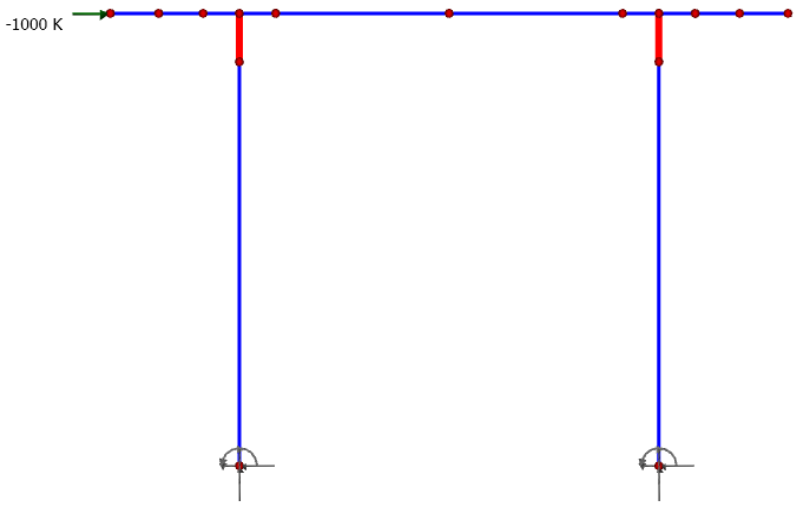
Assume bent foundation piles are in competent material such that foundation flexibility contributes less than 20% to foundation displacement such that soil springs are not required per GS Table 5.3.1-1

Transverse Abutment Springs

Use 50% of Bent Stiffness
in accordance with Caltrans SDC 7.8.2
Δ = 2.656 inch

Bent Stiffness = 1000/2.656 =
Bent Stiffness = 377 kip/in

Transverse Abutment Stiffness = 189 kip/in



Bent Model

Longitudinal Abutment Springs

Use Earthquake-Resisting System (ERS) with Abutment Contribution (Case 2) *GS 5.2.3.2*

Height of Backwall H_W = 60.5 structure depth + 1.50 bearing = 62.0 inch = 5.17 ft
Width of Backwall W_W = 38.75 ft between shear keys

Assume dense sand cohesionless soil

Passive Pressure = $2H_W/3$ = (2)(5.17)/3 = 3.45 ksf *GS Section 5.2.3.3.1*
$P_P = p_P H_W W_W$ = (3.45)(5.17)(38.75) = 690 kip *GS Eqn 5.2.3.3-1*

D_G = Width of Gap between Superstructure and Backwall at Seat Abutment = 1.0 inch
F_W = 0.01 *AASHTO LRFD Table C3.11.1-1*

$$K_{eff1} = \frac{P_p}{(F_w H_W + D_G)} = \frac{690}{[(0.01)(5.17) + 0.083]} = 5114\ kip/ft$$

Column Stiffness = 407 kip/ft each column Based upon Cantilever L = 31.0 ft E_C = 4,347 ksi I_{CR} = 6.454 ft^4
Combined Stiffness = 5,943 kip/ft
Using W = 2,825 kips results in T = 0.764 sec Displacement = 5.56 inch
ARS Force = 2,744 kips Spring Force = 2,367 kip Spring Stiffness = 1,491 kip/ft

Iterate spring stiffness in accordance with GS 5.2.3.3.2 and use K = 1,000 kip/ft
Combined Stiffness = 1,814 kip/ft
Using W = 2,825 kips results in T = 1.382 sec Displacement = 10.31 inch
ARS Force = 1,558 kips Spring Force = 859 kip Spring Stiffness = 804 kip/ft OK within 30% of initial estimate

Seismic Design Parameters

Parameter		Section
Performance Criteria	Life Safety – No Collapse	3.2
Earthquake Resisting System	Type 1 – Ductile Substructure with Essentially Elastic Superstructure	3.3
Analysis Procedure	Procedure 2 – Elastic Dynamic Analysis	Table 4.2-1&2
Design Methods for Displacement	Conventional Ductile Response	4.7.1
Local Displacement Capacity	Nonlinear Static Procedure (Pushover)	4.8.2

Seismic Ground Shaking Hazard

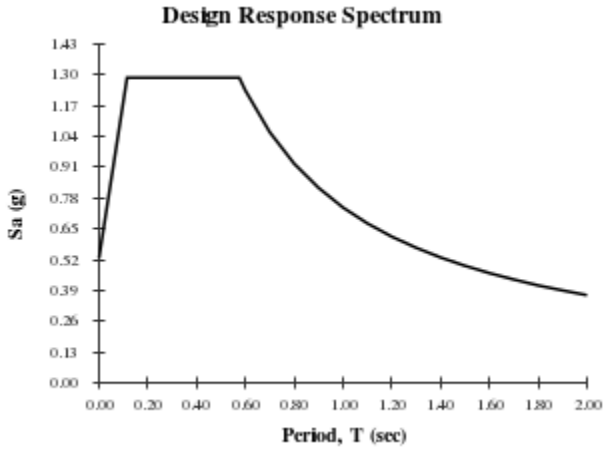
Reference: *GS Section 3.4*

A site-specific hazard analysis is not required as bridge is assumed to be not critical or essential nor does the assumed site consist of Class F soils.

Assumed Parameters from general procedure using USGS/AASHTO Seismic Hazard Maps

Site Class = D

PGA = 0.526g	F_{PGA} = 1.00	A_S = 0.526g
S_S = 1.288g	F_A = 1.00	S_{DS} = 1.288g
S_1 = 0.490g	F_V = 1.510	S_{D1} = 0.740g

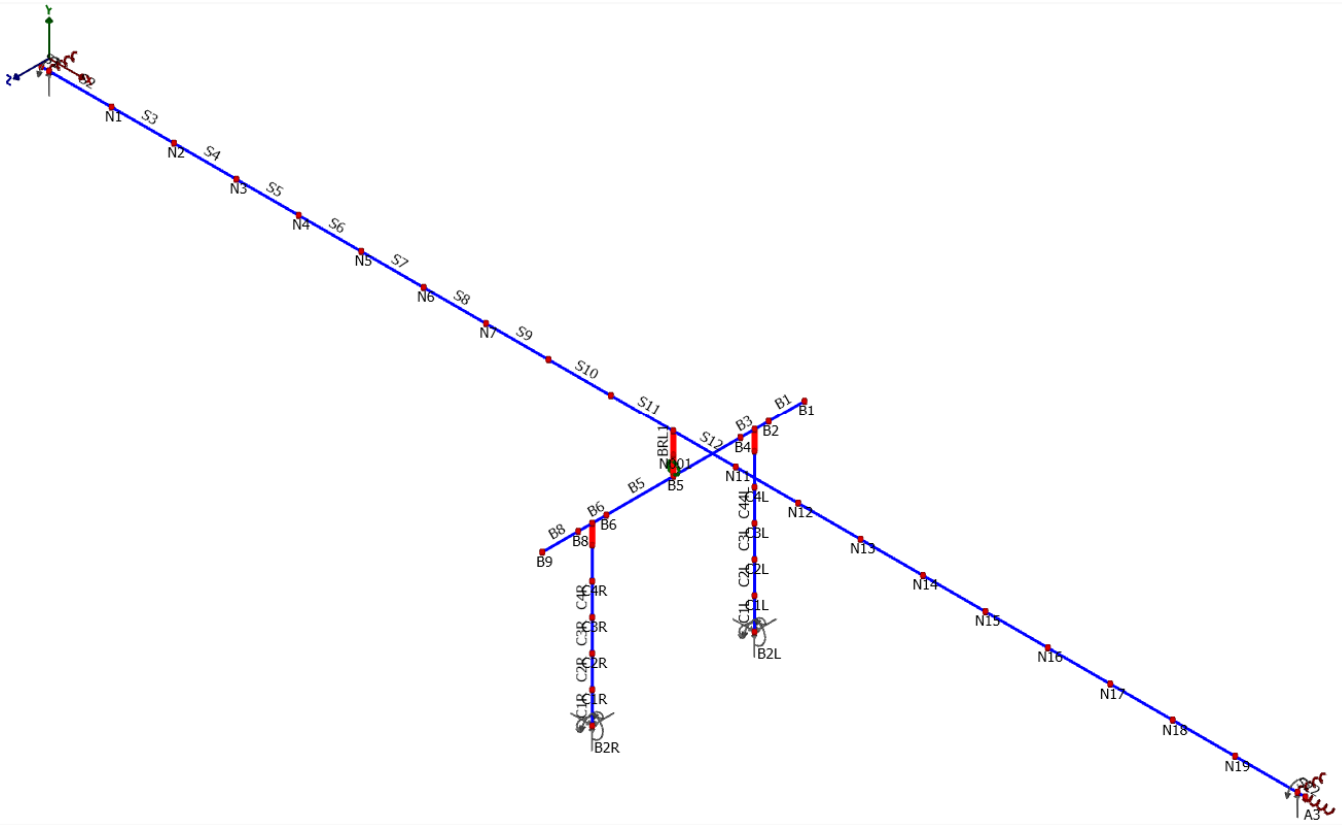


Seismic Design Category = D *GS Table 3.5-1*

Response spectrum applied in in Global X and Z directions using CQC Method and 5% damping

Vertical ground motions are not considered as permitted by GS 4.7.2 as structure is assumed to be not essential or critical

Finite Element Global Model



Modeling Notes

1. Superstructure frame members are located center of gravity located at 1.76 ft from top of deck
2. Superstructure is divided into 10-10.0 ft segments in each span
3. Superstructure includes 1.25 ft cantilever at each abutment
4. Bent 2 cap framing member located at center of gravity located at 8.04 ft from top of deck
5. Rigid link (L= 3.28 ft) connects superstructure to node representing top of bent cap (5.04 ft from top of deck)
6. Rigid link connects node at center of gravity of bent cap to node representing top of bent cap (5.04 ft from top of deck) and this rigid link has Moment Z released at upper end of rigid link to represent the pin connection between superstructure and bent cap
7. Column members are divided into 5-5.0 ft segments
8. Bent 2 column bases are fixed at top of footing (36.04 ft from top of deck) resulting in a total column length of 25 ft
9. Abutments are restrained against vertical displacements and torsion.
10. Z direction (transverse) springs are applied at each abutment
11. X direction (longitudinal) spring is applied at Abutment 3 to represent soil-structure interaction.

Modal Analysis

A modal analysis was performed to obtain 10 mode shapes. Mass includes self weight plus added dead load of barriers and diaphragms.

Modal Results/Participation

Mode Shape Name	frequency (Hz)	Period (Seconds)	X Part	Y Part	Z Part	X Mass K	Y Mass K	Z Mass K
Mode Shape Set 1 Mode 1	0.7402	1.351	0.974	0.000	0.000	2766.920	0.000	0.000
Mode Shape Set 1 Mode 2	1.693	0.5907	0.000	0.000	0.983	0.000	0.000	2794.974
Mode Shape Set 1 Mode 3	2.062	0.485	0.000	0.000	0.000	0.000	0.000	0.000
Mode Shape Set 1 Mode 4	2.962	0.3376	0.000	0.000	0.000	0.124	0.000	0.000
Mode Shape Set 1 Mode 5	4.035	0.2478	0.000	0.702	0.000	0.000	1993.906	0.000
Mode Shape Set 1 Mode 6	9.044	0.1106	0.000	0.028	0.000	0.000	78.212	0.000
Mode Shape Set 1 Mode 7	10.04	0.0996	0.000	0.000	0.000	0.000	0.000	0.022
Mode Shape Set 1 Mode 8	11.74	0.0851	0.000	0.000	0.000	0.756	0.000	0.000
Mode Shape Set 1 Mode 9	15.4	0.0649	0.012	0.000	0.000	33.190	0.000	0.000
Mode Shape Set 1 Mode 10	18.04	0.0554	0.000	0.033	0.000	0.000	93.124	0.000
Mode Shape Set 1 Totals			0.986	0.762	0.983	2800.989	2165.242	2794.996

Mode 1 is longitudinal response
Mode 2 is transverse response

Modal participation > 90% X and Z direction *GS 5.4.3*

Comparison of Results of Conventional Construction to Precast Column Pocket Connection

The only significant difference in the two systems is the width of the bent cap that increased from 6.50 ft to 7.00 ft with a corresponding increase in dead load and stiffness

Result	Conventional	Pocket
Δ_D Longitudinal (inch)	7.392	7.418
Δ_D Transverse (inch)	4.240	4.261

The capacity of the Pocket Connection System is similar to the Conventional Construction System and adequate.

Capacity Protected Members

Bent caps and footings shall be designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity M_{PO}. The components shall have adequate shear and moment capacity based upon the expected material properties. The design of these elements is not included.

Summary of Design

Seismic Parameter	Example 1	Example 2	Example 3	Example 4	Example 5
	Conventional	Grouted Coupler	Swaged Coupler	Grouted Duct/Headed Coupler	Pocket
Longitudinal					
Δ_D (inch)	7.392	7.432	7.416	7.392	7.418
Δ_C (inch)	12.158	9.937	10.862	11.729	12.158
Δ_D/Δ_C	0.608	0.748	0.683	0.630	0.610
μ_D	2.540	2.567	2.554	2.540	2.549
Transverse					
Δ_D (inch)	4.240	4.273	4.263	4.240	4.261
Δ_C (inch)	7.231	5.429	6.296	6.904	7.231
Δ_D/Δ_C	0.586	0.787	0.677	0.614	0.589
μ_D	2.803	2.832	2.831	2.803	2.817
Shear					
V_{PO} (kip)	524	501	503	524	524
ϕV_N (kip)	596	594	594	596	596

Examples 2, 3 and 4 include the use of mechanical couplers in the plastic hinge zone. The proposed guide specifications include provisions to calculate a reduced plastic hinge length, L_P^{SP} for spliced bars. The reduced plastic hinge length results in reduced displacement capacities. Columns spliced with rigid and long couplers will show the lowest displacement capacities. The spliced plastic hinge length for the grouted couplers in Example 2 is 73% of the plastic hinge length L_P for unspliced bars used in the conventional construction Example 1. The swaged couplers and headed couplers have a spliced plastic hinge length of 84% and 95% of L_P respectively. The presence of mechanical couplers required the relocation of the column longitudinal reinforcement to maintain the required 2.0 inch cover. The repositioning of the column longitudinal reinforcement resulted in a slight reduction in the available plastic curvature.

The challenge for the pocket connections will be the design of the bent cap and footings to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity. The limited available space for reinforcement makes the moment and joint shear provisions difficult to attain. As a result, the cap beam width in Example 5 was increased from 6.50 feet to 7.00 feet.