Seismic Pier Design for Steel Pipe Pile Extensions with Concrete Cap Beam

State of Alaska

Department of Transportation & Public Facilities

Bridge Section
Overview

- The purpose of this document is to assist in the design and detailing of Multiple Column / Pile Extension Piers

- The basis of this document is founded on the “Full-Scale Test of a Three Column / Pier Cap Bridge Substructure System Under Simulated Seismic Loading” by Seible, et al.
Typical Pier

Figure 1
Step 1

- Collect the dead load forces in the pier cap and columns due to structure self weight, asphalt, utilities, etc

- These forces should include:

  P - axial
  $V_y$ and $V_z$ - shear
  $M_y$ and $M_z$ - moment
Step 2

- Collect the seismic forces in the pier cap and columns from multimodal computer analysis or other methods

- These forces should include:

  P - axial
  V_y and V_z - shear
  M_y and M_z - moment

- Consider both Load Combination I (100%L + 30%T) and Load Combination II (100%T + 30%L)
Step 3

- Determine the combined axial, shear and moment forces

- Note that the response modification factor, $R$, applies only to seismic moments in ductile members (i.e. where plastic hinges form)

\[
M_{\text{design}} = \sqrt{\left(\frac{\text{Meq-z}}{R} + M_{\text{dl-z}}\right)^2 + \left(\frac{\text{Meq-y}}{R} + M_{\text{dl-y}}\right)^2}
\]

\[
V_{\text{design}} = \sqrt{\left(\text{Veq-z} + V_{\text{dl-z}}\right)^2 + \left(\text{Veq-y} + V_{\text{dl-y}}\right)^2} < V_p
\]

\[
P_{\text{design}} = P_{\text{dl}} \pm P_{\text{eq}}
\]
Step 4

- Using the worst case load combination, determine the amount of longitudinal reinforcement required in the column, $A_{sc}$

- Do not over reinforce the column - this will lead to more cap beam and joint reinforcement

- Use $\phi$ factors as defined in AASHTO
Step 4

- There are many computer programs to aid in the design of concrete columns,
  
  Recol_M  Imbsen & Associates
  ULTCOL  Washington DOT

- Print out the P-M interaction information for later use in the cap beam design

- Note that AASHTO specifies a column reinforcement ratio

  \[ 1\% < \rho < 4\% \]

  but 3\% is a practical upper limit due to joint reinforcement limitations
Step 5

- The ultimate applied shear, $V_{ult}$, is the minimum of either the design EQ shear or the shear associated with plastic hinging of the column, $V_p$.

- Include the column overstrength factor for the concrete “gap” portion of 1.3 and 1.25 for the steel pipe when calculating $V_p$.

- If the required moment capacity of the column is close to the balance moment, use the balance moment in subsequent calculations.
Step 5

The shear associated with plastic hinging is calculated as shown below. *It is good practice to use the $V_p$ for design if practical*

$$V_p = \frac{M_1 + M_2}{H_e}$$

where:

$M_1$ = moment at top of column
$= M_n \times 1.3$ - concrete column

$M_2$ = moment at bottom of column
$= 1.25 \times M_p$ - steel pipe

$H_e$ = effective height of column
$= H + l_m$ (see figure 1)
Step 6

- Determine the size and pitch of the spiral in the column

\[ V_{\text{ult}} < \phi \times V_n \]

where:
\[ \phi = 0.85 \text{ (16th)} \quad 0.9 \text{ (LRFD)} \]
\[ V_n = \text{nominal shear capacity} \]
see AASHTO code or UCSD shear design equations

\[ D = \text{column} / \text{pile diameter} \]
Step 7

- Determine the amount of the cap beam steel required noting that:
  - the height of the cap beam, $h_b$, must be greater than the development length of the column longitudinal steel and
    \[ D < h_b < D \times 1.15 \]
  - the width of the cap beam, $b_j$, must satisfy the following:
    \[ D + 12 \text{ in} < b_j < D + D/2 \]
  - Use the maximum overstrength moment of the column to “load” the cap beam (i.e., $M_p$ at $P_{\text{max}}$)
Step 7

- The required development length of the longitudinal column reinforcement is:

\[ l_d = 0.025 \cdot d_b \cdot F_y / \sqrt{f'_c} \]

where:
- \( d_b \) = diameter of bar
- \( F_y \) = rebar yield strength (psi)
- \( f'_c \) = concrete strength (psi)

- To use this length, welded hoop or spiral reinforcement must be used in the joint (defined in step 9)

- Always extend longitudinal column bars to the top of the cap beam
Step 7

\( \phi \cdot M_n > M_{ult} \)

Where: \( \phi = 0.9 \) for bending

\( M_n = \) nominal bending capacity

\( M_{ult} = M_p + M_{dl} \)

\( M_p = \) plastic moment moment capacity of column associated with \( P_{max} \)

Check that the cap beam is not over or under reinforced and that temperature and shrinkage steel requirements are satisfied
Step 8

- Determine the size and spacing of shear stirrups required in the cap beam

\[ V_{\text{ult}} < \phi \times V_n \]

where:

\[ \phi = 0.85 \text{ (16th ed.)} \quad 0.9 \text{ (LRFD)} \]

\[ V_n = V_c + V_s \]

\[ V_{\text{ult}} = V_{\text{dl}} + V_{\text{p-cap}} \]

\[ V_{\text{p-cap}} = \text{shear in cap beam due to plastic hinging of column} \]

\[ = \frac{1.5 \times M_p}{S_{\text{col}}} \text{ (approx.)} \]

- Use shear at “d” from face of column
Step 9

- Determine the size and spacing of welded hoops required in the joint region of the cap beam

- This steel is needed to provide development length and confinement for the column longitudinal steel

\[ s = \frac{4A_h}{D'\rho_s} < \frac{h_b}{4} \]

where:
- \( s \) = welded hoop spacing
- \( A_h \) = area of welded hoop
  - e.g. #5 hoop = 0.31 in\(^2\)
Step 9

- Continued

\[ l_a = \text{anchored length of } A_{sc} \]
\[ A_{sc} = \text{Area of column longitudinal steel rebar} \]
\[ h_b = \text{height of cap beam} \]
\[ \lambda_o = \text{overstrength factor} \]
\[ = 1.4 \]
\[ \rho_s = 0.3*\lambda_o*A_{sc} / l_a^2 > 3.5*\sqrt{(f'_c)/F_y} \]
\[ D' = \text{core diameter of column} \]

- Provide a cap beam height greater than the anchorage length required for the column longitudinal steel (see step 7)
Typically these bars will be field welded after placement
Step 10

- Determine the average principal tensile stress in the joint

\[
p_{c,t} = \frac{(f_v + f_h)}{2} \pm \sqrt{\left(\frac{(f_v - f_h)}{2}\right)^2 + v_j^2}
\]

where:
\[
f_v = \frac{P_c}{b_j^* (D + h_b)}
\]
Step 10

Continued

\[ f_h = \frac{V_c}{b_j h_b} \]
\[ v_j = \frac{M_c}{h_b D b_j} \]

- \( M_c \) = moment in the column
- \( V_c \) = shear in the column
- \( P_c \) = axial load in the column
- \( D \) = column diameter
- \( h_b \) = height of cap beam
- \( b_j \) = width of cap beam and
  - \(< \sqrt{2} \times D \)
  - \(< D + h_b \)

- Always check your signs (+/-)
Step 10

- Use $M_c$, $V_c$, and $P_c$ which result in maximum principal tension, $p_t$

- If the principal tension ($p_t$) is greater than $3.5\sqrt{f'_c}$ then additional joint reinforcement is required - that is:

  If $p_t < 3.5\sqrt{f'_c}$ then done

  If $p_t > 3.5\sqrt{f'_c}$ then provide the additional reinforcement defined in the following steps

  If $p_t > 15\sqrt{f'_c}$ then joint will not work - try different pier geometry
Strut and Tie Model

- The model developed by UCSD (shown below) was used to generate the following joint design procedure.

- Area of steel to resist tensile forces ($T_{es}$, $T_{bb}$ and $T_c$) is determined from joint geometry and reinforcement pattern.
Step 11

- Determine the extra amount of shear reinforcement (paired hoops) required outside the joint region, $A_{vj}^o$

- Space the stirrups evenly in a region equal to the cap beam height. Total area $A_{vj}^o$ to each side of the column

$$A_{vj}^o > 0.125 * \lambda_o * A_{sc}$$

where:
$A_{sc} = \text{area of column}$
$\lambda_o = \text{overstrength factor} = 1.4$
Shear Reinforcement Outside Joint

\[ A_{vj}^o > 0.125 \times A_{sc} \times \lambda_o \]

Put the paired hoops on each side of the joints
Step 12

- Determine the amount of shear reinforcement (paired hoops) required in the joint, $A_{ivj}^i$

- Space these bars evenly within the joint region over the column

$$A_{ivj}^i > 0.095 \times \lambda_o \times A_{sc}$$

where:

$A_{sc} =$ area of column longitudinal steel

$\lambda_o = 1.4$
Shear Reinforcement Inside Joint

\[ A_{vJ} > 0.095 \times A_{sc} \times \lambda_o \]

Space paired hoops evenly within joint region
Step 13

- Additional top and bottom longitudinal reinforcement is required to develop the joint strut-and-tie mechanism.

- Add the following amount of cap beam longitudinal steel, $\Delta A_b$, *in addition to what is required to resist bending alone*, to both top and bottom:

  \[
  \Delta A_b > 0.17 \times \lambda_o \times A_{sc}
  \]

where:

$A_{sc} =$ area of column longitudinal steel

$\lambda_o = 1.4$
Additional Longitudinal Beam Reinforcement

\[ \Delta A_b > 0.170 \times A_{sc} \times \lambda_o \]

Put additional longitudinal bars on top AND bottom
Step 14

- Provide seismic “J” bars within the joint regions to prevent buckling of the longitudinal steel in the cap beam and to provide additional confinement of the joint region

- Two or three “J” bars per longitudinal cap beam bar should be adequate for most cases

- Space the bars evenly within the joint so as to prevent buckling of longitudinal cap beam bars
Transverse Seismic “J” Bars

Place two or three three “J” bars per longitudinal cap beam bar within joint region

Could use welded, headed bars if desired
Detailing Notes

- Provide concrete core down pipe pile below the depth of effective fixity (point of maximum moment) by at least 3 pile diameters or to the point where the pile moment is about half the maximum moment.

- Make sure that the longitudinal cap beam bars are fully developed - may need to provide 90° hooks.

- Use headed reinforcement in place of the “J” bars and on the ends of the longitudinal cap beam bars if space is tight.
Detailing Notes

- Use paired shear stirrups (hoops) in pier cap beams. This provides better confinement of concrete and a more even distribution of steel within joint region to better carry the loads.

- Generally, more smaller bars are better than few larger bars for serviceability. However, you must still meet bar spacing requirements for concrete placement.

- Although the earthquake load case often governs the pier design, you must still examine the other load combinations (strength and serviceability).
References

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- **AASHTO** AASHTO LRFD Bridge Design Specifications (1996)
- **ASCE-ACI Committee 445** Recent Approaches to Shear Design of Structural Concrete (1998)
- **Silva, Sritharan, Seible, and Priestley** Full-Scale Test of the Alaska Cast-in-Place Steel Shell Three Column Bridge Bent (1999)