C6.6 Piers

C6.6.1.1.1 Frame pier

15 March 2012

In the past the office generally has preferred tapered cantilevers for frame pier caps, but the office now prefers constant depth cantilevers. Because pier caps for frame piers are typically formed with studded plywood rather than steel forms any change to the cantilever geometry should not increase formwork costs significantly. The benefits of tapered frame pier caps over rectangular frame pier caps are generally limited to a moderate reduction in concrete quantity and a moderate aesthetic advantage.

The benefits of a rectangular frame pier cap over a tapered frame pier cap are more extensive:

- Simplification of forming for the end of the pier cap, including the cap support system.
- Elimination of the bent bars and variable shear stirrups at the end of the cap. This in turn reduces issues with maintaining reinforcing clearance at the form face. Additionally, the number of bar marks will be reduced.
- Easier layout and tying of reinforcement. Elimination of the end tapers will simplify assembly of the reinforcement cage.
- Simplification of pier cap design.

2011 ~ Selection of Frame or T-Pier

These changes refer back to BDM 6.6.2.6 for the vehicular collision force policy. AASHTO is likely to be changing the policy in 2011, and the office can better manage its response if the policy is in one location only.

C6.6.1.1.1. T-pier

2011 ~ Selection of Frame or T-Pier

These changes refer back to BDM 6.6.2.6 for the vehicular collision force policy. AASHTO is likely to be changing the policy in 2011, and the office can better manage its response if the policy is in one location only.

C6.6.1.2. Vehicular collision force

22 May 2012

Because of significant changes to the vehicular collision force guidelines in the 2012 AASHTO LRFD Specifications the office decided to make the AASHTO changes (with minor modifications) in the July 2012 Bridge Design Manual release.

07 December 2016 ~ Collision design requirements for frame piers without crash walls

The office has updated its policy with respect to collision design requirements for multi-column frame piers without crash walls.

Some sources of information regarding the policy include TTI research reports and policies from several other state DOTs particularly Wisconsin, Florida, and Texas as listed below.

- C.E. Buth, W.F. Williams, M.S. Brackin, D. Lord, S.R. Geedipally, A. Abu-Odeh, Analysis of Large Truck Collisions with Bridge Piers: Phase 1. Report of Guidelines for Designing Bridge Piers and Abutments for Vehicle Collisions, Texas Transportation Institute (TTI) Technical Report 9-4973-1, May 2010 (http://tti.tamu.edu/publications/catalog/record/?id=31734)
- C.E. Buth, M.S. Brackin, W.F. Williams, G. Fry, Collision Loads on Bridge Piers: Phase 2. Report of Guidelines for Designing Bridge Piers and Abutments for Vehicle Collisions, Texas Transportation Institute (TTI) Technical Report 9-4973-2, March 2011 (http://tti.tamu.edu/publications/catalog/record/?id=31735)
- Wisconsin DOT considers solid shafts and hammerhead pier shafts adequately designed for collision based on their standard designs. Multi-columned piers require a minimum of three columns, minimum 4.0 feet

column diameters, 1% minimum vertical column reinforcement based on gross section, and #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. [Bridge Manual, Article 13.4.10, July 2016, Wisconsin DOT, http://wisconsindot.gov/dtsdManuals/strct/manuals/bridge/ch13.pdf]

- Florida DOT considers the vehicular collision force as a point load on the pier column with no distribution of force due to frame action within the pier, foundation, and superstructure. No further analysis of the piles, footings, pier cap, or other columns is required. The column shear capacity is checked assuming failure along two shear planes inclined at 45 degrees above and below the point of force application. [Structures Design Guidelines, Volume 1, Article 2.6.8, January 2017, Florida DOT, http://www.fdot.gov/structures/StructuresManual/CurrentRelease/Vol1SDG.pdf]
- Texas DOT allows the column to be designed for only the shear created by the collision force and does not consider the transfer of the force to other elements such as bent caps, footings, piles, or drilled shafts. Conservatively the shear capacity of the column can be determined using a β of 2.0 and θ of 45 degrees without consideration of axial loads. Piers with three or more columns may use two shear planes to distribute the collision force. Additionally single column piers with a gross-sectional area of 40 square feet and a least dimension of no less than 5 feet with column transverse reinforcement of at least #4 ties at a 12 inch pitch are assumed to meet the collision force requirements. [Bridge Design Manual – LRFD, Chapter 2, Section 2, October 2015, Texas DOT, http://onlinemanuals.txdot.gov/txdotmanuals/lrf/lrf.pdf]

Consider the 600 kip vehicular collision force per LRFD as a point load acting on the pier column. Neglect any distribution of force due to frame action within the pier, foundation and superstructure. Further analysis of the piles, footings, pier cap, other columns, etc., is not required. Check the column shear capacity assuming failure along two shear planes inclined at θ above and below the point of force application. Under this design procedure the impacted structure is expected to remain stable and to continue to support the bridge superstructure subsequent to the collision event.

The Iowa DOT design for the column shear capacity is based on the MCFT method in AASHTO LRFD 2014 Article 5.8.3.4.2.

28-day concrete strength, $f'_c = 4.0$ ksi
Yield strength of shear reinforcement, $f_y = 60.0$ ksi

Column diameter = 48.0"
Shear reinforcement bar size = #5
Shear reinforcement spacing = 4.0"
Longitudinal reinforcement bar size (assumed) = #11
Cover to shear reinforcement = 2.0"

From AASHTO LRFD 2014 Article C5.8.2.9:

b_v = column diameter = 48.0"
D = column diameter = 48.0"
D_r = diameter of circle passing through the centers of the longitudinal reinforcement
= 48.0" - (2)*(2.0" + 0.625" + 0.5*(1.410")) = 41.34"
$d_e = (D/2) + (D_r)/\pi = (48.0"/2) + (41.34"/\pi) = 37.16"$
$d_v = 0.9d_e = (0.90)*(37.16") = 33.44"$

Assume $\varepsilon_s = 0.006$ in/in
$\beta = 4.8/(1 + 750\varepsilon_s) = 4.8/(1 + (750)*(0.006 \text{ in/in})) = 0.873$
$\theta = 29 + 3500\varepsilon_s = 29 + (3500)*(0.006 \text{ in/in}) = 50$ deg

Concrete shear resistance
$V_c = 0.0316\beta(f'_c)^{0.5}b_v d_v = (0.0316)*(0.873)*((4.0 \text{ ksi})^{0.5})*(48.0")*(33.44") = 88.56$ kips

Shear reinforcement resistance
Assume $\alpha = 90$ deg
$V_s = (A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha)/s$

= (2 legs)*(0.31 in^2)*(60.0 ksi)*(33.44")*(cot(50 deg) + cot(90 deg))*(sin(90 deg))/(4.0") = 260.95 kips

One shear plane

$V_n = V_c + V_s$ = 88.56 kips + 260.95 kips = 349.51 kips

Two shear planes

V_n = (2)*(349.51 kips) = 699.02 kips

Assume ϕ_v = 0.90 per AASHTO LRFD 2014 Article 5.7.2.1, not Article 1.3.2.1

$\phi_v V_n$ = (0.90)*(699.02 kips) = 629.11 kips > 600 kips

C6.6.1.3. Earthquake

4 June 2008 ~ Summary

The AASHTO LRFD seismic requirements were made considerably more complex in the 2008 interim. The 2007 specifications varied the restrained horizontal connection design forces in Seismic Zone 1 as either 0.1 or 0.2 times the tributary load, but the 2008 interim sets the horizontal connection force as either 0.15 or 0.25 times the tributary load. The 2008 interim also removes an elastomeric bearing requirement that clouded the use of a friction coefficient of 0.2.

2007 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there were 10 non-blank pages organized in what seemed to be a logical sequence.

- The design process depended on the Acceleration Coefficient, A. For Iowa, A varied between 0.02 for northeastern Iowa to 0.053 in southwestern Iowa [AASHTO-LRFD Figure 3.10.2-2].
- Because all Iowa Acceleration Coefficients did not exceed 0.09, all of Iowa was in Seismic Zone 1 [AASHTO-LRFD Table 3.10.4-1].
- Site Coefficients, S, varied from 1.0 for Soil Profile Type I (best profile) to 2.0 for IV (worst profile).
- For Seismic Zone 1 with A ≤ 0.025 and Soil Profile Type I or II the horizontal design connection force in the restrained direction was 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For all other cases in Seismic Zone 1 the horizontal design connection force in the restrained direction was 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For elastomeric bearings a design coefficient of friction could be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD C14.8.3.1]. That assumption was clouded by another rule regarding friction at the strength limit state [AASHTO-LRFD 14.7.6.4] (which was removed in the 2008 Interim).

2008 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there are 42 non-blank pages, many of which are maps.

- The seismic design process depends on three coefficients determined from maps and three factors determined from tables. Two coefficients and two factors are applicable to Seismic Zone 1 requirements.
- Seismic Zone 1 is defined by an acceleration coefficient, $S_{D1} < 0.15$, which is determined by $S_{D1} = F_v S_1$ [AASHTO-LRFD 2008 Table 3.10.6-1, 3.10.4.2]. From the map for Iowa, S_1 varies from 0.019 in northwestern Iowa to 0.044 in extreme southeastern Iowa [AASHTO-LRFD 2008 Figure 3.10.2.1-3]. F_v varies from 0.8 for Site Class A to 3.5 for Site Class E [AASHTO-LRFD 2008 Table 3.10.3.2-3]. Site Class is defined for different soil types and layers from A to F [AASHTO-LRFD 2008 Table 3.10.3.1-1]. Site Class F is defined for peat, very high plasticity clays, or more than 120 feet of soft/medium stiff clays. For Site Class F a site specific analysis is recommended.
- All of Iowa, with the possible exception of a few sites in extreme southeastern Iowa (Lee County) generally would be classified as Seismic Zone 1 as follows: northwestern Iowa S_{D1} = (0.019)(3.5) = 0.0665 maximum (except Site Class F); extreme southeastern Iowa S_{D1} = (0.044)(3.5) = 0. 154 maximum (except Site Class F). Extreme southeastern Iowa sites with Site Class E or F soil profiles could be considered Seismic Zone 2 under a strict interpretation of the AASHTO LRFD Specifications.

- The forces to be applied to bearing connections in Seismic Zone 1 depend on the short period acceleration coefficient, $A_s < 0.05$, which is determined by $A_s = F_{pga}PGA$ [AASHTO-LRFD 2008 3.10.9.2, 3.10.4.2]. From the map for Iowa, PGA varies from 0.015 in north central Iowa to 0.040 in southeastern Iowa. F_{pga} is to be taken for the applicable Site Class [AASHTO-LRFD 2008 Table 3.10.3.2-1].
- The A_s in Iowa will vary above and below 0.05 depending on Site Class. Therefore, the horizontal design connection force in the restrained direction will be either 0.15 or 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 2008 3.10.9.2]. In general the 0.15 factor will apply for northern Iowa and the 0.25 factor will apply in southern Iowa, but the designer will need to check for the specific bridge site.
- For elastomeric bearings a design coefficient of friction may be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD 2008 C14.8.3.1]. Generally if not restrained, steel reinforced elastomeric bearings in Seismic Zone 1 need not be anchored other than by friction [AASHTO-LRFD 2008 C3.10.9.2]. However, before making the final decision regarding anchorage, the designer shall consider the rules in the steel reinforced elastomeric pads article in the manual [BDM 5.7.4.2].

C6.6.2.12 Uniform temperature

C6.6.2.12.3 Out-of-plane forces

17 January 2003 ~ Policy discussion with Assistant Bridge Engineer

There have been two guidelines in the office for determining the locations of axial fixity for piles. The older guideline is from the 1979 version of "Design Criteria for Piers":

> The pile length used to determine the footing rotation is assumed to be 50% and 75% of the pile length for timber and steel piling respectively.

The more recent guideline is the following:

> The percent of the pile length used to determine the footing rotation is assumed to be:
> (1) 50% when most of the pile capacity is due to friction bearing; generally wood, prestressed concrete, or steel piles not driven to bedrock, or
> (2) 75% when most of the pile capacity is due to end bearing, generally steel piles driven to bedrock.

For current design, use the more recent guideline. It is based on two concepts. The first concept is that a friction pile will transfer its load to the soil over the entire length of the pile, and thus axial deformation effectively will occur over half the pile length. The second concept is that an end bearing pile typically will transfer some load by friction and thus will not deform axially over its full length but over more length than a friction pile. The 75% value is a judgment factor.

C6.6.1.4. Shrinkage

9 May 2008 ~ Summary

Generally, it is recognized that creep relieves a part of the stresses caused by shrinkage in restrained members but that it may be necessary to reinforce for shrinkage stresses. The AASHTO LRFD Specifications do not cover shrinkage in frames directly. However, for prestressed structures the specifications indicate that the designer need only consider initial elastic deformation for columns in monolithic frames and that other members will be subjected to reduced forces due to creep. Although there are no simple recommendations for shrinkage of frame piers, it is reasonable to use the AASHTO 28-day shrinkage coefficient of 0.0002 for design forces and provide the usual shrinkage and temperature reinforcement. Even though the shrinkage forces computed for 0.0002 will be reduced by the AASHTO LRFD load factor in strength limit state combinations, shrinkage forces will be included in every limit state rather than in only three load combinations under the AASHTO Standard Specifications.

Shrinkage research and standards

New York researchers (Antoni and Beal, 1971) instrumented a bridge pier to verify the AASHO 1964 specification requirements for coefficient of thermal expansion, temperature variation, and shrinkage. There were difficulties with

the instrumentation, and the researchers could not separate temperature effects from shrinkage effects, but the researchers concluded that the 0.0002 shrinkage coefficient was reasonable.

In recent years there has been much study of concrete shrinkage in the hope that cracking could be reduced or eliminated. In overlays and composite structures the existing structure generally has undergone most or all of its shrinkage, and the new concrete is intended to be bonded to that structure and yet undergo shrinkage without cracking. Because of the infinite variety of concrete mixtures, the wide variety of structural conditions, the related factors such as thermal expansion/contraction and creep, and the differences between tension and compression creep, researchers have not found any simple and definitive answers to questions regarding shrinkage.

One researcher, however, has suggested that at two years creep reduces sustained tensile stresses by about one-third (Alexander 2005, 2007). The researcher also suggested that the designer provide reinforcement for tension caused by shrinkage.

The AASHO/AASHTO Standard Specifications have included the concrete shrinkage coefficient of 0.0002 since 1941. Specifications of that vintage had no commentary, so the source of the coefficient is unknown, except that New York researchers attribute the coefficient to experimental data from small laboratory specimens (Antoni and Beal, 1971).

The ISU strip seal report (Bolluyt et al. 2001) concluded that 0.0002 was an appropriate shrinkage value for concrete decks on prestressed beams or steel girders, although some of the bridges studied indicated more or less shrinkage. The report did not recommend any modification factors. (There was no proposal to use modification factors with thermal expansion/contraction either, but there was a recommendation to increase the temperature range for computing concrete bridge thermal movements.)

Of the surrounding six states, only Nebraska (0.0002) and Wisconsin (0.0003) include concrete shrinkage in their strip seal designs. In the ISU report there is no indication of whether the states include shrinkage in frame pier design.

The AASHTO LRFD equations for shrinkage and compressive creep [AASHTO-LRFD 5.4.2.3] show that both develop in the same pattern and, assuming the strain in concrete is about 10% of the crushing strain (among other assumptions), the creep and shrinkage curves approximately match. Using the shrinkage equation with assumptions, shrinkage can be computed of about 0.0007 at one year and 0.0008 at 2.5 years. The AASHTO LRFD Specifications, however, generally indicate shrinkage of 0.0002 at 28 days and 0.0005 at one year [AASHTO-LRFD 5.4.2.3.1].

The AASHTO LRFD Specifications do require that effects of prestressing deformations on adjoining elements of the structure be evaluated [AASHTO-LRFD 5.9.2]. These deformations could be considered to be similar to shrinkage deformations. Additionally the specifications state the following:

> In monolithic frames, force effects in columns and piers resulting from prestressing the superstructure may be based on the initial elastic shortening.
>
> For conventional monolithic frames, any increase in column moments due to long-term creep shortening of the prestressed superstructure is considered to be offset by the concurrent relaxation of deformation moments in the columns due to creep in the column concrete.
>
> The reduction of restraining forces in other members of a structure that are caused by the prestress in a member may be taken as….

Considering the statements above, it is prudent to include some value of shrinkage in design. The statement regarding use of initial elastic shortening in monolithic frames could be taken to suggest the shrinkage coefficient of 0.0002 at 28 days. The next AASHTO statement could be taken to suggest that relaxation due to creep will relieve additional column moments due to shrinkage after 28 days. The AASHTO slowly imposed deformation equation (not copied above) with assumptions indicates that a 100-kip force, such as a cap force based on shrinkage, for time from 28 days to 896 days would reduce to 43 kips, less than half the initial force.

References

Alexander, S.J. (2007). "The Importance of Time in Understanding Concrete Behavior," *Conference on Structural Implications of Shrinkage and Creep of Concrete, SP-246*, pp 283-289.

Alexander, S.J. (2005). "Managing Deflection, Shortening and Cracking Arising from Restrained Contraction," *Shrinkage and Creep of Concrete, SP 227*, pp 1-20.

American Association of State Highway Officials (AASHO). (1941) *Standard Specifications for Highway Bridges, Third Edition,* American Association of State Highway Officials, Washington, DC.

Antoni, C.M. and D.B. Beal. (1971). *Temperature and Shrinkage Stresses in a Concrete Bridge Pier, Research Report 69-6*, Engineering Research and Development Bureau, New York Department of Transportation, Albany, New York.

Bolluyt, J.E., V.B. Kau, and L.F. Greimann. (2001). *Performance of Strip Seals in Iowa Bridges, Pilot Study, Final Report Project TR-437,* College of Engineering, Iowa State University, Ames, Iowa.

C6.6.3.2 Limit states

2010 ~ Service I Limit State

With use of LEAP Bridge Concrete Substructure (formerly RCPIER) software the Service I load combination may appear to control pile design, however there is no need to consider Service I for strength design of piles. Service I may be used to check settlement of piles.

C6.6.4.1 Frame piers and T-piers

2010 ~ Location of fixity for frame and T-piers on pile footings

Column analysis and design is sensitive to slenderness, and the designer should not model a column taller than the structural configuration allows. Although it would be acceptable to model a pier column as fixed at 2 feet below top of footing or at mid-depth of footing, the column is restrained significantly at the top of footing, and the designer should assume fixity at that elevation to minimize slenderness. Available pier software makes it relatively easy to analyze alternate models, and for pile and footing design the designer should assume pier columns extend to bottoms of footings.

C6.6.4.1.1 Pier cap

C6.6.4.1.1.1 Analysis and design

April 2013 ~ Guidelines for mass concrete

Previous guidelines for mass concrete have been misinterpreted due to aesthetic shapes and unusual configurations. The revised statement in this article

> "… when the least dimension of any element exceeds 4.5 feet".

is intended to identify the large volumes of concrete that need to be considered mass concrete. There may be cross sections along a tapered cap that meet the above definition of mass concrete and those that do not, but the sections that meet the definition classify the cap as mass concrete, and therefore require the mass concrete provisions.

2011 ~ Longitudinal Reinforcement for Shear

Equation 5.8.3.5-1 will compute additional longitudinal reinforcement anywhere along a member where there is non-zero factored shear. For a typical frame pier cap, LEAP Bridge Concrete Substructure (formerly RCPIER) will provide the area of the additional reinforcement at sections along the cap selected by the engineer.

The AASHTO LRFD Specifications, however, permit the designer to neglect the additional reinforcement at certain locations as follows:

> "The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or the load introduces direct compression into the flexural compression face of the member."

For direct loading this exception is shown graphically in AASHTO LRFD Figure C5.8.3.5-2 extending a distance $d_v \cot\Theta$ either side of the maximum moment. For the typical frame pier cap with two cantilevers, the exception will apply at each support column and at each beam bearing where M_u is maximum (V_u changes sign) in the cap. The designer should make use of this exception.

2010 ~ Longitudinal Reinforcement for Shear

The requirement in the AASHTO LRFD Specifications for longitudinal reinforcement for shear [AASHTO LRFD 5.8.3.5] was added to the specifications at the same time as modified compression field theory. The requirement substitutes for ACI Code and AASHTO Standard Specifications rules for end supports, points of inflection, and cutoffs in tension zones. Because AASHTO LRFD Figure C5.8.3.5-2 does not show a cantilever condition, designers have asked whether the requirement would increase the top steel in pier cap cantilevers. Numerical examples indicated that the increase could be significant.

Examples in the literature are inconclusive, but a cantilever truss analogy worked in the office similar to the simple span truss analogy in *Reinforced Concrete Mechanics and Design, Fifth Edition* by Wight and MacGregor showed that the top steel in a pier cap cantilever need not be increased beyond that required for moment at the pier column. As in AASHTO LRFD Figure 5.8.3.5-2 the demand for longitudinal reinforcement did increase away from the support over the entire distance to the free end. For a typical pier cap cantilever design without bar cutoffs the top reinforcement is not affected.

12 April 2012

Torsion on a pier cap can be developed in a number of ways such as:

- Some degree of fixity between the pier cap and the superstructure,
- An eccentric bearing line or eccentric bearing lines with respect to the centerline of the cap,
- Lateral loads acting at the top of the cap, or
- Unbalanced vertical dead and live load on two sets of bearing lines. This will cause torsion whether or not there is an expansion joint at the pier.

The usual reinforcement cage for a pier cap includes longitudinal corner bars and regularly spaced transverse hoops. This cage typically will have a small amount of torsional resistance available beyond flexural and shear resistance and, as a result, the designer may use judgment to avoid checking torsion for a typical pier cap.

C6.6.4.1.2 Pier column

C6.6.4.1.2.1 Analysis and design

April 2013 ~ Guidelines for mass concrete

Previous guidelines for mass concrete have been misinterpreted due to aesthetic shapes and unusual configurations. The revised statement in this article

> "… when the least dimension of any element exceeds 4.5 feet".

is intended to identify the large volumes of concrete that need to be considered mass concrete. There may be cross sections along a tapered column that meet the above definition of mass concrete and those that do not, but the sections that meet the definition classify the column as mass concrete, and therefore require the mass concrete provisions.

2011 ~ Minimum T-Pier Column Reinforcement

In order to limit longitudinal reinforcement in large pier columns, designers in the past have been permitted to consider a reduced effective cross section area, if that reduced area had a minimum of 1% reinforcement. To be in compliance with the AASHTO LRFD Specifications in 2001, the office additionally restricted the minimum reinforcement to 0.7% of the actual gross area.

Since 2001 Article 5.7.4.2 in the AASHTO specifications has been revised to consider f'_c and f_y in setting the minimum reinforcement for the gross area. Equation 5.7.4.2-3 now limits the minimum reinforcement to 0.79% of the gross column area for $f'_c = 3.5$ ksi and $f_y = 60$ ksi. For $f'_c = 4$ ksi and $f_y = 60$ ksi the minimum reinforcement must be 0.90% of the gross area. In some cases, even with one of these reductions, there is difficulty in fitting the longitudinal reinforcement at the perimeter of a large pier column.

For bridges in Seismic Zone 1 the current AASHTO LRFD Specifications also permit a reduced effective column area provided that the reinforcement is the larger of 1% for the reduced area or the amount from Equation 5.7.4.2-3 for the reduced area. However there is no minimum limit for the reduced effective area. Based on previous successful experience the office will allow the designer to design for a reduced effective area that as a minimum is 50% of the actual area. This minimum area follows ACI 318-08 Article 10.8.4.

2010 ~ Structural Models for Piers

Initially with the use of LEAP Bridge Concrete Substructure (formerly RCPIER) the office recommended that designers model piers with columns fixed 2 feet below the top of footing. This modeling applied for both pier frame and foundation and made it possible to run the software only once. However, this modeling resulted in columns 2 feet taller than they actually were and, in some cases, led to a significant increase in moment magnification.

In order to minimize moment magnification the office now is giving designers the option of fixing the columns at the tops of the footings. This option will require the designer to run the software twice, once for the pier columns and cap and once for the footings and piles.

2010 ~ Pier Column Shear Reinforcement

Pier columns subjected to significant lateral loads need to be designed for shear as well as axial loads. The text of the manual was revised to alert the designer to the need to meet both shear stirrup and column tie requirements.

C6.6.4.1.3 Pier footing

C6.6.4.1.3.1 Analysis and design

2010 ~ Structural Models for Piers

The option of using two models in LEAP Bridge Concrete Substructure (formerly RCPIER) reduces the effect of moment magnification for design of pier columns. See C6.6.4.1.2.1 for additional information.

21 March 2001 / Revised 29 January 2003 ~ Punching Shear and Wide Beam Shear (For LRFD neglect references to AASHTO Standard Specifications.)

Pre-LRFD the manual gave rules for checking piles at design and check scour conditions under the AASHTO Standard Specifications. The rules were based on service load design (SLD) and have been updated for the AASHTO LRFD Specifications. The following is a brief summary of the LRFD specifications regarding scour.

Service limit state

- 10.5.2: Consider foundation movements, including movement at the design scour condition.
- 10.5.5.1: Use a resistance factor of 1.0 for deflection at the design scour condition.
- 10.7.2.1: Evaluate overall stability for loss of support due to scour.

Strength limit state

- 10.5.3.1: Consider loss of support due to scour at the design flood.

- 10.5.5.2.1: Factored foundation resistance after design flood scour must be greater than factored load with scoured soil removed.
- 10.5.5.2.3: Use resistance factors specified for geotechnical resistance, structural resistance, and drivability analysis.
- 10.7.3.6: Select pile penetration to be adequate after scour. Consider debris loads during flood event.

Extreme event limit state

- C10.5.4: Design for check flood scour.
- 10.5.5.3.2: Nominal resistance ($\varphi = 1.0$) after check flood scour is to be adequate for unfactored strength limit state loads. For uplift take $\varphi = 0.8$ or less. Consider debris loads during the check flood.
- 10.7.4: Use check flood and resistance factors from 10.5.5.3.2.

For geotechnical design the former procedures are superseded by the AASHTO LRFD Specifications, which are more liberal for check scour. After severe flooding, scour will be evaluated by Iowa DOT bridge inspection teams under the Scour Watch program, and therefore, under check scour conditions, safety will be assured by design, field inspection, and bridge closures.

For structural design, Structural Resistance Level - 2 (analogous to 9 ksi) at KL/r of 80 for design scour has been taken as the basic condition and extrapolated under LRFD to Structural Resistance Level – 3. For check scour the maximum slenderness has been taken at 120 (the maximum for a main compression member) for Structural Resistance Levels 1 to 3. This will result in a small apparent margin of safety for piles under check scour, more than required by the AASHTO LRFD Specifications, which will allow some capacity for bending of the pile under stream flow pressure. If the superstructure is partially or fully inundated by the 100-year flood the designer will be required to check the 500-year flood condition at the extreme event limit state as discussed in BDM 6.6.2.7.

For Table 6.6.4.1.3.1-1 the maximum slenderness ratios for design scour were determined to provide an apparent margin of safety, γ/φ (load factor/resistance factor), of about 3.4 for a compression load. The maximum slenderness ratio of 120 [AASHTO-LRFD 6.9.3] at the check scour condition results in an apparent margin of safety of about 1.4, minimum, for compression load. Because the basic column stability formula was changed from the AASHTO Standard Specifications to the AASHTO LRFD Specifications, the apparent margins of safety in the two specifications cannot be compared directly.

Steel H-piles for Structural Resistance Level 4 shall be fully analyzed and designed for design and check scour conditions.

References

Galambos, T.V., Ed. (1998). *Guide to Stability Design Criteria for Metal Structures, Fifth Edition,* John Wiley & Sons, Inc., New York, NY.

Tide, R.H.R. (2001). “A technical note: derivation of the LRFD column design equations,” *Engineering Journal,* third quarter, 137-139.

17 May 2018 ~ Frame pier and T-pier pile footings with shallow bedrock requiring rock coring

Rock coring for frame pier and T-pier footings may be required for pile stability when there is less than 10 feet of soil between the bottom of the pier footing and top of bedrock. Confirm with the Chief Structural Engineer the necessity and feasibility of using rock coring for pile footings under this situation.

The designer should carefully consider whether battered piles are required for piers with rock coring. Prebored holes for battered piles are more likely to require casing to maintain the hole. Drilling and coring operations are also more difficult for battered piles.

In the text below the term rock socket is specifically reserved for the bottom 3 feet of the rock core that is filled with concrete in order to lock the piles in place. Any rock coring above the bottom 3 feet is referred to as cored rock prebore. Any prebore of the soil above the top of bedrock is referred to as soil prebore.

The soil prebore and any cored rock prebore shall have a minimum diameter 4 inches greater than the maximum cross-sectional dimension of the pile. The rock socket shall have a minimum diameter 2 inches greater than the maximum cross-sectional dimension of the pile. [Example: For HP10 steel piles specify a minimum of 18 inches for the diameter of prebored holes in soil and rock and a minimum of 16 inches for the diameter of rock sockets.]

The minimum depth of the rock socket shall be 3 feet into sound rock. The piles shall be seated by driving the pile to the target driving resistance. A minimum of 3 feet of Class C structural concrete shall be placed in the bottom of the rock socket to lock the base of the piles in place. The concrete may be placed before or after the piles are seated. The contractor shall make a reasonable effort to clean the bottom of the socket before inserting pile or placing concrete. If piles are seated after the concrete is placed, then a retarder may be required to ensure the concrete remains plastic while the piles are driven. The contractor shall brace the piles in the correct position until the concrete achieves 4 ksi compressive strength. The remaining portion of the holes shall be backfilled with granular material.

When rock coring is required the designer shall generally indicate the following on the Longitudinal Section Along Centerline Approach Roadway of the Situation Plan sheet:

- Type, size, and length of pile
- Bottom of pier footing elevation
- Bottom of prebored hole elevation, diameter of prebored hole, and length filled with granular material
- Anticipated top of rock elevation
- Minimum bottom of rock socket elevation, socket diameter, and length filled with concrete

The General Notes shall reference rock coring notes on the pier sheets. For example:

> THE BRIDGE CONTRACTOR SHALL PREBORE HOLES FOR PILES AT PIERS _____. HOLES SHALL BE BORED TO THE ELEVATIONS SHOWN ON THE "LONGITUDINAL SECTION ALONG CENTERLINE APPROACH ROADWAY" ON DESIGN SHEET _____. SEE PIER PILING NOTES ON DESIGN SHEET _____ FOR SPECIAL ROCK CORING REQUIREMENTS.

The following notes or similar ones shall be included with the pier plan sheets:

> THE _____ INCH DIAMETER PREBORED HOLE IN THE SOIL SHALL EXTEND FROM THE BOTTOM OF THE PIER FOOTING TO THE TOP OF THE _____ STRATUM AS SHOWN ON THE SOIL PROFILE SHEET WHICH IS ANTICIPATED AT ELEVATION _____. THE _____ INCH DIAMETER ROCK SOCKET SHALL BE CORED TO ELEVATION _____ OR LOWER SUCH THAT THE BOTTOM OF THE SOCKET SHALL BE AT LEAST 3 FEET INTO THE _____ STRATUM. THE CONTRACTOR SHALL CLEAN THE BOTTOM OF THE ROCK SOCKET BEFORE INSERTING PILES OR PLACING CONCRETE.
>
> SEAT THE PILING IN THE BEDROCK BY DRIVING IT TO THE TARGET DRIVING RESISTANCE. THE NUMBER OF HAMMER BLOWS SHALL BE LIMITED TO PREVENT DAMAGE TO THE PILING. THE SOCKET SHALL BE BACKFILLED WITH 3 FEET OF CLASS "C" STRUCTURAL CONCRETE BEFORE OR AFTER DRIVING PILES. CONCRETE PLACED BEFORE PILES ARE DRIVEN SHALL REMAIN PLASTIC UNTIL PILE DRIVING IS COMPLETE. RETARDER MAY BE REQUIRED AS DIRECTED BY THE ENGINEER. PILES SHALL BE BRACED IN THE CORRECT POSITION UNTIL CONCRETE HAS REACHED 4 KSI COMPRESSIVE STRENGTH. THE REMAINING PORTION OF THE HOLES SHALL BE BACKFILLED WITH GRANULAR MATERIAL.
>
> IF BEDROCK IS NOT ENCOUNTERED ABOVE ELEVATION _____, THEN NO ADDITIONAL PREDRILLING IN THE SOIL IS REQUIRED AND NO "CORING ROCK SOCKET" QUANTITY SHALL BE MEASURED FOR PAYMENT. THE PILE SHALL BE DRIVEN INTO ROCK IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS.

When rock coring is specified designers shall include bid item 2599-999909 CORING ROCK SOCKET with units of linear feet. A rock coring bid item reference note similar to the following shall be included:

> THE QUANTITY OF "CORING ROCK SOCKET" IS BASED ON THE ANTICIPATED TOP OF ROCK ELEVATION AND CORING _____ FEET INTO THE ROCK FOR EACH PILE IN PIERS _____. THE NUMBER OF LINEAL FEET OF ROCK CORING WILL BE MEASURED IN THE FIELD AND PAID FOR

AT THE CONTRACT UNIT PRICE BID PER LINEAL FOOT. THE PRICE BID INCLUDES THE COST OF PREDRILLING _____ INCH DIAMETER ROCK SOCKETS A MINIMUM _____ FEET INTO _____. THE FOLLOWING COSTS SHALL BE INCIDENTAL TO THE CONSTRUCTION PRICE: DRILLING OR EXCAVATING TO THE TOP OF THE BEDROCK TO OBTAIN ACCESS FOR CORING, DISPOSING OF ALL EXCAVATED MATERIAL, ANY CASING REQUIRED TO KEEP HOLE OPEN, FURNISHING AND PLACING CLASS "C" STRUCTURAL CONCRETE TO BACKFILL THE SOCKET TO THE REQUIRED ELEVATION, AND BACKFILLING THE HOLES FROM THE TOP OF SOCKET CONCRETE TO THE BOTTOM OF PREBORE ELEVATION WITH GRANULAR MATERIAL. IF PIER PILES CAN BE DRIVEN TO A MINIMUM _____ FEET OF LENGTH, NO MEASUREMENT OR PAYMENT WILL BE MADE FOR "CORING ROCK SOCKET".

C6.6.4.1.4 Seal coat

For this LRFD manual the loads, resistances, and factors were calibrated directly from service load design (SLD). Computations and checks under either SLD or LRFD should give the same results.

C6.6.4.2.1.1 Steel H-piles

Steel H-pile, pile bent example for structural design only:

Given: Three-span (67'-6, 80'-0, 67'-6) PPCB bridge over gully without stream, 218 feet long
with 40-foot roadway and 15-degree skew
Steel H-piles
Factored total DC + DW + LL + IM load at pile bent = P_u = 1800 kips
Maximum height to underside of non-monolithic cap = 18 feet
Ground slope along pile bent 1:30 maximum
Soil profile: 0-3 feet: top soil
3-75 feet: firm - very firm glacial clay with N = 14

(1) Determine size and number of H-piles for structural condition above ground

Check applicability of Bridge Design Manual simplified method [BDM 6.6.4.2.1]
Bridge length: 218 feet < 250 feet …OK
Roadway width: 40 feet < 44 feet …OK
Skew: 15 degrees < 45 degrees …OK
Thermal expansion for steel substructure [AASHTO-LRFD 3.4.1, p3-11]:
(80/2)(12)(0.000006)(50)(1.0) = 0.144 inches < 0.45 inches …OK
Ground slope: 1:30 < 1:10 …OK

Choose pile shape [BDM Table 6.6.4.2.1.1]
Try HP 12x53 because H = 18 feet (> 16 feet for HP 10x57)

Determine number of piles [BDM Table 6.6.4.2.1.1]
$n = P_u/\varphi_c P_n = 1800/(0.7)(192) = 13.4$, try 14 piles
Spacing = 3'- 0 3/4 > 2.50 feet …OK [spacing from H40-06 series sheet H40-48-06]

(2) Check HP 12x53 structural condition in the ground

Per pile, P_n = 224 kips [BDM Table 6.2.6.1-1]
For bent, $P_r = n\varphi_c P_n = (14)(0.6)(224) = 1881.6$ kips > P_u = 1800 kips …OK

For determining contract length and driving target see the similar examples for steel pipe and prestressed concrete in Track 1, Examples 6 and 7 in LRFD Pile Design Examples ~ 2013.