Seismic Design of Pile-to-Pile Cap Connections in Flexible Pier Structures

By Vitaly B. Feygin, P.E.

Dedicated to my mentor and friend R.J. Mancini, P.E.

Diver inspections in the aftermath of the 1985 Kobe and 2010 Haiti earthquakes uncovered an unexpected discovery. In many locations, piles were completely or partially separated from the pile caps. In some areas, pile sockets were ripped off the pile cap. While such behavior was not anticipated, thorough review of the forces generated at the pile-to-pile cap interface indicated that, on many occasions, connection ductility was grossly overestimated.

Current design practice utilizes an assumption, introduced by J.W. Gaythwaite, that the piles in a pile bent have a fixity point some distance (D) below the mud-line, equal to half the critical length. This is determined from a lateral pile deflection analysis based on the Winkler spring soil model. Gaythwaite produced two equations for calculating the depth between the mud-line and the assumed point of fixity:

- For granular soils, $D=1.8\ (EI/n_h)^{0.2}$ (*Equation 1*)
- For consolidated clay, $D=1.4\ (EI/k_s)^{0.25}$ (*Equation 2*)

Where,
E = modulus of elasticity of pile material;
I = moment of inertia of the pile cross-section;
n_h = horizontal subgrade modulus for granular soils, which varies with depth; and
k_s = modulus of subgrade reaction for clay.

Gaythwaite emphasized that *Equations 1* and *2* apply only if the total pile embedment length exceeds 3D. However, his assumption is only partially correct. What Gaythwaite has identified is nothing more than the zero-deflection point whose partial fixity is represented by a rotational spring with a stiffness defined as follows:

$k_r = M/\Theta$ (*Equation 3*)

Where,
M = flexural moment at zero-deflection point; and
Θ = slope of the elastic curve at zero-deflection point.

To produce a partial fixity support condition, the pile embedment length should be sufficient to develop at least two zero-slope points within the soil medium. The arbitrary 3D embedment length introduced by Gaythwaite sometimes falls short of that requirement.

G.P. Tsinker suggested another model utilizing non-linear springs for pile soil supports; however, such springs utilizing P-y curves were only recently introduced into some finite element analysis software packages. Linear Winkler springs traditionally used for pile bent analysis frequently place the zero-deflection point significantly higher on the piles, underestimating soil crushing. Obviously, the use of non-linear soil springs increases the complexity of the pile bent analysis. However, non-linear soil supports better predict forces at the pile-to-pile cap interface.

Understandably, the complexity of the analytical procedure greatly affects design price. Nevertheless, deficient assumptions often impact the *ultimate* price of the product, adding the cost of remedial repairs required in the aftermath of a destructive event. Oversimplification of design assumptions frequently delivers an inferior product to the client.

Fundamentals of Seismic Forces

There are two basic types of seismic waves: body waves and surface waves.

Body waves travel along rays extended from the earthquake's epicenter, deep under the earth surface, to the surface of the earth. They have two independent wave components:

- P-waves, called primary longitudinal waves or "compression waves." These travel in compression motions with speeds approaching 16,000 ft/sec in solid rock.
- S-waves, called secondary waves or "shear waves." These travel along the same ray path as compression waves but cause sinusoidal ground displacements perpendicular to the direction of wave propagation. The speed of S-waves is about 50-60% of the speed of P-waves in the same soil medium.

Similar to body waves, surface waves have two independent components: Rayleigh waves and Love waves. Once surface waves are activated by body waves, they become independent, propagating along the earth's surface.

- Rayleigh waves travel as ripples with a speed comparable to that of S-body waves. Their behavior is similar to that of waves on the surface of water, creating vertical rolling motions in the direction of propagation. Soil particles in a Rayleigh wave move on an elliptical trajectory in a direction opposite to wave propagation. Rayleigh waves have a low frequency, but long duration, and comparatively small initial amplitude. However, the effects of the Rayleigh waves can be compared to those of tsunami waves during their final stage, when they are gaining amplitude in shallow waters. In some geotechnical conditions, surface waves are just as devastating, quickly gaining amplitude within very short distances, depending on the reflective and absorptive characteristics of the underlying soil medium.

continued on next page

Vitaly B. Feygin, P.E., is a principal structural engineer with Marine and Industrial Consultants, with offices in Baltimore and Tampa. He is the author of two patents related to sea walls, composite cofferdams, bridge fenders and port structures. He may be reached at ***(vfeygin.mic@gmail.com)****.*

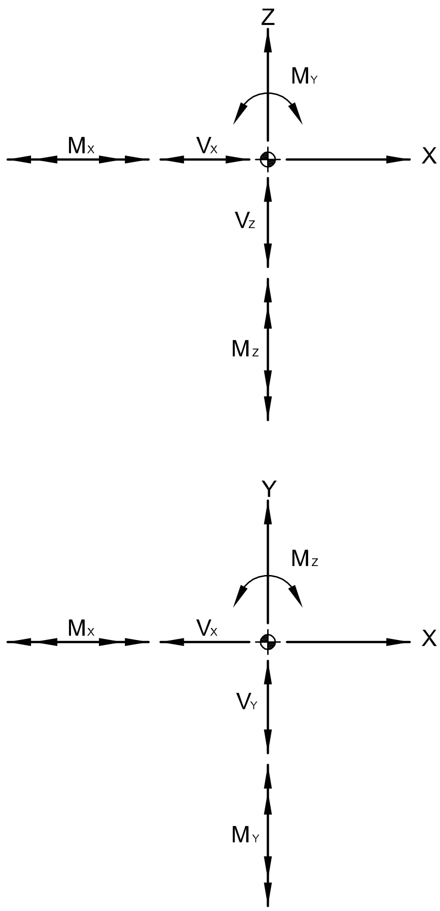


Figure 1: Forces acting on the pile head at any time during seismic event.

- Love waves cause horizontal shifts perpendicular to the direction of wave propagation. They have the highest amplitude.

Frequently, a direct surface wave can create a reflective wave traveling in the opposite direction. Superposition of two harmonic waves depends on the relative phase of each wave. Superposition of two waves traveling in opposite directions can create a local standing wave with amplitude equal to the sum of two individual wave amplitudes.

Such a phenomenon is based on soil medium reflection and transmission characteristics. The sign of the reflected wave depends on the reflection boundaries of the soil medium. Prediction of the phases of direct and reflected waves is a demanding and nearly impossible task. However, engineers cannot ignore the possibility of a standing wave. Destruction of the port waterfront facilities in the aftermath of the 2010 Haiti earthquake strongly suggests the presence of standing waves during that seismic event.

A brief review of the nature of seismic forces indicates that, at any given time during an earthquake, a point on the earth's surface is constantly moving in all six degrees of freedom. Therefore, accounting for forces restricting pile head movement in all six degrees of freedom is important for the successful design of ductile pile-to-pile cap connections.

Analytical Procedure for Pile Bent Analysis

Equations 1 and *2* provide a first trial approximation of the zero-deflection point. A more exact location can be determined by a finite element analysis of the pile bent. In some software packages, pile lateral supports can be modeled as non-linear springs from soil P-y curves. Locating the zero-deflection point along the pile embedment length allows a designer to establish an effective pile length. Pile unsupported length is taken as the length between the pile cap and the zero-deflection point. Since piles are slender compression elements experiencing a combination of compression, shear and flexural forces, the pile slenderness ratio as described by *Equation 4* carries great importance:

$\lambda = k_e L_u / r$ (*Equation 4*)

Where,

k_e = effective column length;

L_u = pile unsupported length; and

r = least radius of gyration of the pile cross section.

The design value for k_e can be determined from Jackson-Moreland alignment charts, utilizing the relative rotational stiffness at both ends of the pile. Rotational stiffness at the pile-to-pile cap connection can be easily established, while rotational stiffness at the zero-deflection point is provided by *Equation 3*. The slenderness ratio is used for preliminary sizing of piles in the pile bent and should be kept below 100.

Design for Seismic Event

Determining base shear acting on the pile head provides a value for the magnitude of the force in one direction only. Chapter 12.5 of ASCE 7-05 recommends the design of foundation components for 100% of the dynamic forces in one direction acting simultaneously with 30% of the forces in the perpendicular direction. Structures should be analyzed in both major directions, and pile connections should satisfy the most critical case.

Pile-to-pile cap connections experience forces in all six degrees of freedom. While there is clarity among designers as to how determine the M_x, M_z, V_x, V_z, and V_y components (*Figure 1*), ASCE 7-05, AASHTO and IBC are silent on the planar torsional component M_y. It would be prudent to assume pile torsional fixity at the pile-to-pile cap interface, with torsional force applied at the level of the zero-deflection point.

Depending on the pile length, piles are characterized as short, intermediate or long by *Equation 5*:

$K = L/(EI/f_v)^{1/5}$ (*Equation 5*)

Where,

L = pile embedment length;

E = modulus of elasticity of shaft material;

I = moment of inertia of the "beam"; and

f_v = modulus of subgrade reaction of the soil medium.

Piles with K > 4 are classified as long piles. In long piles, the so-called "partial fixity point" is the first zero-deflection point along the pile embedment length developing at least two zero-slope points. Such fixity is described by the rotational spring of *Equation 3* *(see page 21)*.

Some traditional designs based on full fixity at the zero-deflection point greatly underestimate flexural moment at the pile-to-pile cap interface. That was likely one of the reasons why some connections failed during extreme seismic events. An additional factor was something that is often completely ignored by pier designers. Unfortunately, all applicable codes are silent on the effect of Love waves on the pile-to-pile cap connections.

The Love component of the surface wave can twist the structure in plan. Therefore, pending further research, the following torsional force at the pile-to-pile cap interface is suggested:

$My = 0.125 V_{BS} * (n*S)^2 * y * d_p / I_p$ (*Equation 6*)

Where,

V_{BS} = base shear acting on the pile bent, disregarding reduction due to ductility of the lateral force resisting system;

S = spacing between the pile bents (deck span);

n = number of deck spans within ½ of the Love wave length;

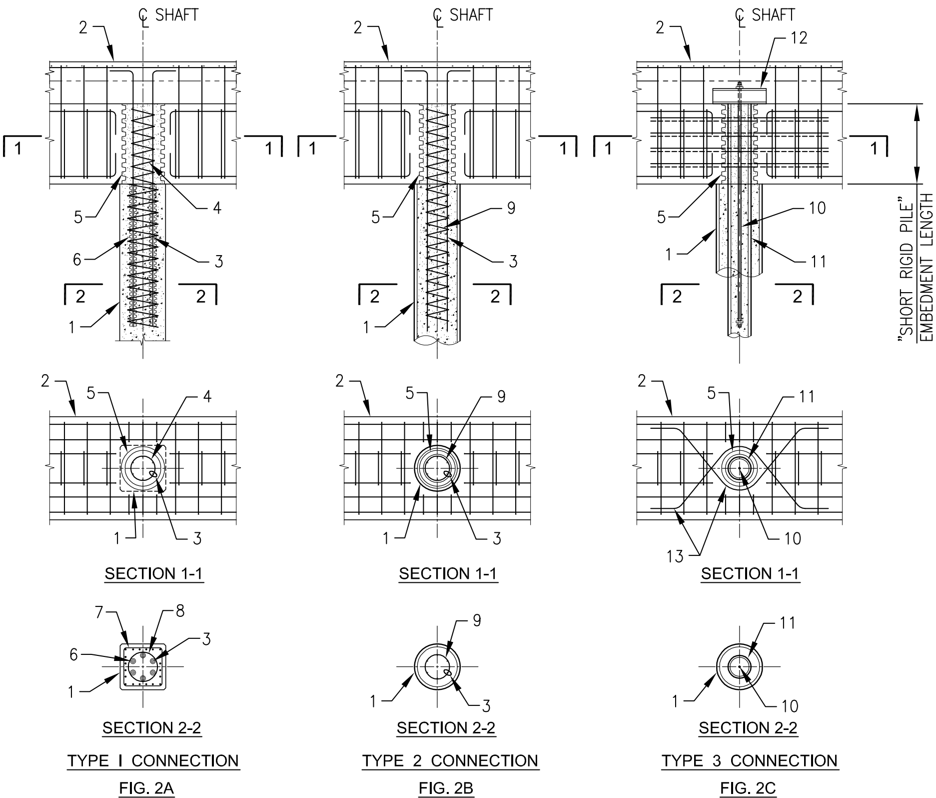
y = the distance between the c.g. of the bent and extreme pile of the bent;

d_p = pile diameter; and

I_p = polar moment of inertia of the piles in (n-1) pile bents.

Review of Pile-to-Pile Cap Connection Details

There have been several attempts made to solve the problem of pile-to-pile cap connection failures. The recent work of M. Teguh, C.F. Duffield, et al. indicates that current



1. PRECAST PILE OR STEEL PIPE PILE
2. PILE CAP
3. DOWEL OF THE "SHORT PILE"
4. SPIRAL OF THE "SHORT PILE" WITHIN PILE CAP
5. SOCKET SLEEVE
6. DOWEL SLEEVE
7. SPIRAL OF THE PRECAST PILE
8. SPIRAL OF THE "SHORT PILE" WITHIN PRECAST PILE
9. SPIRAL OF THE "SHORT PILE"
10. THREADED ROD
11. ANCHOR PILE
12. BACK TO BACK ANCHOR CHANNELS
13. HORIZONTAL Ω – STIRUP

international design practice results in joint details with congested steel reinforcement, while formation of the plastic hinge in the pile remains a serious risk. The paper rightfully states that "lack of careful detailing and poor confinement of core concrete" were at the root of the problem, pointing to inelastic damage that occurred at the pile cap interface during recent catastrophic seismic events. Current design practice does not provide designers with a tool to perform accurate analysis of a pile-to-pile cap connection's physical behavior during an earthquake.

The following details provide engineers with a simple and yet reliable tool for the design of a ductile pile-to-pile cap connection. This connection should be treated as a "short pile" embedded into a very stiff medium (reinforced concrete). *Figures 2a, 2b* and *2c* explain the concept of pile-to-pile cap connection design by reviewing several types of such details. Note that development of a plastic hinge at the pile-to-pile cap interface does not typically result in failure of the structure; that only occurs when a plastic hinge develops within the pile socket of the pile cap.

Type 1: Connection between precast pile and pile cap (*Figure 2a*). This detail shows rebar dowels grouted into special sleeves within the precast pile. Dowels are anchored into the closure pour of the pile cap. A portion of the dowel cage is embedded into the pile cap sleeve. Dowels are confined by ⅜-inch-diameter spiral whose pitch is debatable. There are arguments in favor of a 6-inch spiral pitch in a "short rigid pile" stub and arguments in favor of a reduced value, but there is no evidence that pitch of the spiral is a significant factor influencing ductility.

Something that is a factor is the ductility of the pile socket confinement. In connections where this becomes a critical element, the designer is urged to use closely spaced Ω-shaped stirrups as shown in the Type 3 connection (*Figure 2c*). The effect of Ω-shaped stirrups is explained below.

Type 2: Regular connection between steel pipe pile and pile cap (*Figure 2b*). This detail shows an arrangement very similar to that of Type 1.

Type 3: Improved pile connection detail for zones with strong seismic activity and connections subject to high seismic effects (*Figure 2c*). Forces acting on the pile head are shown in the diagram in *Figure 1*. To understand the design requirements for pile head connections, the designer should review all mechanisms restricting pile head movement in all six degrees of freedom.

continued on next page

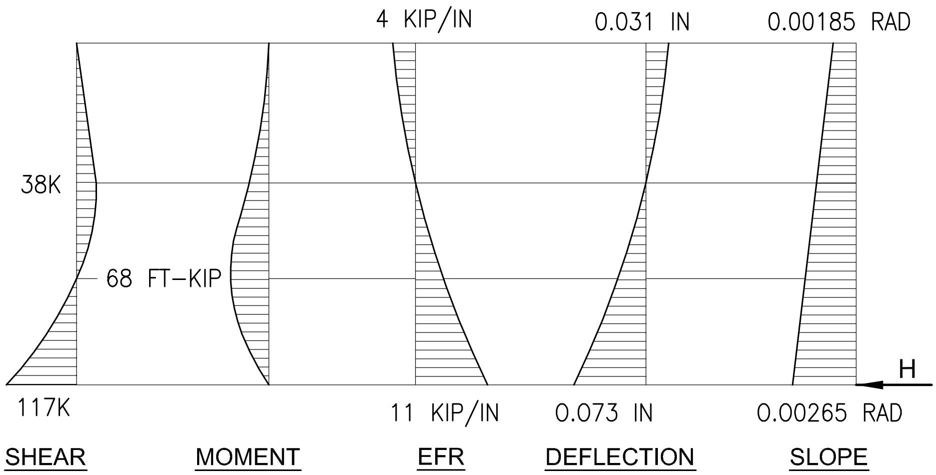


Figure 3: Force and deflection diagrams.

A pile dowel cage confined by a spiral is viewed as a "short rigid pile" in a very stiff medium. Analysis is simplified by the fact that the P-y curve for concrete is a well-known parameter. The designer can easily establish upper boundaries for an elastic foundation reaction curve using maximum passive pressure along the "pile" length as a limiting value. Ω-shaped stirrups can significantly increase the effective width of the elastic foundation, increasing the pile cap's shear capacity. Coupled Ω-shaped stirrups provide effective anchorage of the sleeve into the compression zone of concrete.

The effective width of the unreinforced sleeve elastic foundation, b_{eff}, tends to be equal to the sleeve diameter d_{slv}. The effective width of the reinforced sleeve elastic foundation is defined as:

$b_{eff} = d_{slv} + b - 2d'$ (*Equation 7*)

Where,

d_{slv} = sleeve diameter;

b = width of the pile cap; and

d' = concrete cover.

To model a short stiff pile correctly, the designer should select the minimum pile stiffness allowing a straight deflection line, or a slope curve with nearly constant slope along the pile length.

Figure 3 shows shear, V; moment, M; elastic foundation reaction, EFR; deflection and slope diagrams of the "short pile" in one direction. Similar forces are acting in the orthogonal direction. For "short pile" analysis, forces from both directions should be combined as vectors. Based on that analysis, the designer should check dowel reinforcement for a combination of direct tension, flexural, shear and torsional forces; bearing stress on the concrete confining the pile socket; deflection of the socket; slope of the short pile within the socket; and flexural moment developed in the "short rigid pile". It is important to remind designers that all tension forces caused by flexure should be algebraically combined with tension forces caused by shear and torsion.

A connection can be considered satisfactory if all conditions listed below are satisfied:

- The combined stress in any dowel or pipe section of the "short pile" does not exceed the yield stress of the steel;
- The bearing stress under the "short pile" effective width footprint does not exceed the bearing capacity of the concrete (EFR / b_{eff});
- The crushing of concrete inside of the socket does not exceed 1⁄16-inch ("short rigid pile" deflection);
- The slope of the "short pile" is described by nearly straight line; and
- The flexural moment developed in the "short rigid pile" can be resisted by the short pile flexural reinforcement (Type 1 and Type 2 connections), or a short pile pipe section (Type 3 connection).

Analysis of Type 1 and Type 2 connection details can explain pile embedment reinforcement shear failure at pile-to-pile cap interfaces during seismic events similar to the 1995 Kobe and 2010 Haiti earthquakes. The Type 3 detail provides a better alternative and addresses another reported failure mechanism – rupture of the pile socket.

Ductility of the concrete confining the pile socket becomes a serious issue in regions with high seismic activity. Since a seismic wave has a composite multispectral and multidirectional nature, it is easy to imagine a simultaneous downward force and lateral force acting at the pile-to-pile cap interface, normal to the pile bent frame. Such a force combination can rip off the pile socket from the pile cap. Failures of that nature were observed in the aftermath of the Kobe and Haiti earthquakes.

Placement of closely spaced Ω-shaped stirrups significantly improves the ductility of the socket detail. The size and spacing of Ω-shaped stirrups should be based on forces normal to the pile bent, and a vertical force equivalent to the gravity force tributary to one pile. The shear plan for that failure mode should be taken at the vertical boundaries of b_{eff}.

Ω-shaped stirrups can be used in Type 1 and Type 2 connections as well. While the additional cost of such an improvement is minor, the benefits of such a modification are difficult to ignore.

Summary

Many pile-to-pile cap connection failures in seismically active regions could have been prevented with proper design and detailing. A great deal of research on that subject was done by several groups of engineers and researchers. However, recent failures of pile-to-pile cap connections indicate that previously suggested models were somewhat inadequate. Solutions suggested by this article provide a simple and yet reliable model for analytical investigation of the pile-to-pile cap details. It is evident that moment connection details of Type 1 and Type 2 are viable solutions for regions with low seismic activity and in connections exposed to moderate seismic forces. Connections designed for high seismic forces require the "short pile approach" and the more ductile Type 3 detail.▪

References:

Gaythwaite, John W. *Design of Marine Facilities for Berthing, Mooring and Repair of Vessels*. New York: Van Nostrand Reinhold, 1990.

Tsinker, Gregory. *Port Engineering. Planning, Construction, Maintenance and Security*. New York: John Wiley and Sons, 2004.