$C_{(i)}^{pile}$ = compression force in *i*th pile (kip)

$T_{(i)}^{pile}$ = tension force in *i*th pile (kip)

For SDC B, in cases in which elastic forces control, the axial demand on an individual pile shall be determined according to Eq. 6.4.2-2, with the elastic forces and moments according to Article 4.4 substituted for the plastic hinging forces and moments.

In soft soils, consideration shall be given to the possibility that the pile cap may not dominate the lateral stiffness of the foundation, as is expected in competent soil, possibly leading to significant lateral displacements as pile capacities are mobilized in lateral loading. In soft soils, piles shall be designed and detailed to accommodate displacements-induced head moments and axial forces based on analytical findings.

6.4.3—Lateral Capacity of Pile Foundations

The lateral capacity of pile foundations in soils shall be evaluated. The capacity evaluation shall include the resistance developed by the pile cap and the lateral shear resistance of the piles. The amount of displacement to mobilize the resistance from the cap and the piles shall be considered in the capacity estimate. The Designer shall verify that the geotechnical and structural capacity of the pile cap and the piles exceed the lateral demand transmitted by the columns.

C6.4.3

Lateral capacity of the pile cap should include the passive pressure mobilized at the face of the cap and the interface shear resistance developed along each side of the cap. Procedures used to estimate the passive pressure at the face of the cap can normally involve static passive pressure equations and charts given in Section 3 of the *AASHTO LRFD Bridge Design Specifications.* Wall friction of two-thirds of the friction angle should be used in this determination. The amount of displacement to mobilize the passive pressure should follow guidance given in Section 10 of the *AASHTO LRFD Bridge Design Specifications*.

The shear along the side of the cap can be estimated using the effective pressure at the mid-height of the cap thickness ($_v$), a lateral stress factor (K_o) of 0.5, and the friction angle (ϕ) of the backfill material (i.e., F_s = ($_v$ K_o tan ϕ) A_{surf} where A_{surf} is the surface area for each side of the cap. If a cohesive soil is used for backfill, the undrained strength of the cohesive soil is used in place of $_v$ K_o tan ϕ. The amount of displacement to mobilize the shear capacity along the side of the cap is usually less than 0.5 in. For many cases, the contributions of side shear are small and can be neglected in the capacity estimate.

Methods used to estimate the load-deformation response of piles are established in Section 10 of the *AASHTO LRFD Bridge Design Specifications* and can be used to develop a stiffness value for the pile group. If liquefaction is possible, appropriate adjustments should be made to evaluate stiffness for the liquefied case. This evaluation involves use of the residual strength of the liquefied soils. Because of uncertainties in the development of liquefaction, checks should also be performed for the nonliquefied case to determine the more critical of the two.

6.4.4—Other Pile Requirements

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report. Group reduction factors established in the geotechnical report should be included in the analysis and design of piles required to resist lateral loads. The nominal geotechnical capacity of the piles should be used in designing for seismic loads.

Where reliable uplift pile capacity and the pile-to-footing connection and structural capacity of the pile are adequate, pile side resistance from uplift of a pile footing may be used in the capacity evaluation with the Owner's approval, provided that the magnitude of footing rotation will not result in unacceptable performance according to P-ǎ requirements stated in Article 4.11.5. Additionally, the connection between the footing or cap and the piles should be capacity protected to resist the maximum force the pile could deliver.

All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. Minimum reinforcement shall be in accordance with Article 8.16.

Footings shall be proportioned to provide the minimum embedment, clearance, and spacing requirements according to the provisions of the *AASHTO LRFD Bridge Design Specifications*. The spacing shall be increased when required by subsurface conditions. For SDC D, embedment of pile reinforcement in the footing cap shall be in accordance with Article 8.8.4.

C6.4.4

Friction piles may be considered to have uplift resistance due to skin friction, or, alternately, 50 percent of the ultimate compressive axial load capacity may be assumed for uplift capacity. Uplift capacity need not be taken as less than the weight of the pile (buoyancy considered).

6.4.5—Footing Joint Shear for SDCs C and D

All footing to column moment resistive joints in SDCs C and D shall be proportioned such that the principal stresses meet the following criteria:

Principal compression:

$$p_c \leq 0.25 f_c^{'} \qquad (6.4.5\text{-}1)$$

Principal tension:

$$|p_t| \leq 0.38\sqrt{f_c^{'}} \qquad (6.4.5\text{-}2)$$

in which:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6.4.5\text{-}3)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6.4.5\text{-}4)$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} D_{ftg}} \qquad (6.4.5\text{-}5)$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \tag{6.4.5-6}$$

where:

B_{eff}^{ftg} = effective width of footing (in.)

For circular columns:

$$B_{eff}^{ftg} = \sqrt{2} D_{cj} \tag{6.4.5-7}$$

For rectangular columns:

$$B_{eff}^{ftg} = B_c + D_{cj} \tag{6.4.5-8}$$

and:

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \tag{6.4.5-9}$$

in which:

A_{jh}^{ftg} = effective horizontal area at mid-depth of the footing assuming a 45° spread away from the boundary of the column in all directions as shown in Figure 6.4.5-1 (in.^{2})

For circular columns:

$$A_{jh}^{ftg} = (D_{cj} + D_{ftg})^2 \tag{6.4.5-10}$$

For rectangular columns:

$$A_{jh}^{ftg} = \left(B_c + D_{ftg}\right)\left(D_{cj} + D_{ftg}\right) \tag{6.4.5-11}$$

where:

D_{cj} = column width or diameter parallel to the direction of bending (in.)

B_c = diameter or width of column or wall measured normal to the direction of loading (in)

D_{ftg} = depth of footing (in.)

P_{col} = column axial force including the effects of overturning (kip)

f'_c = uniaxial compressive concrete strength (ksi)

T_c = column tensile force associated with the column overstrength plastic hinging moment, M_{po} (kip)

$\sum T_{(i)}^{pile}$ = summation of the hold down force in the tension piles (kip)

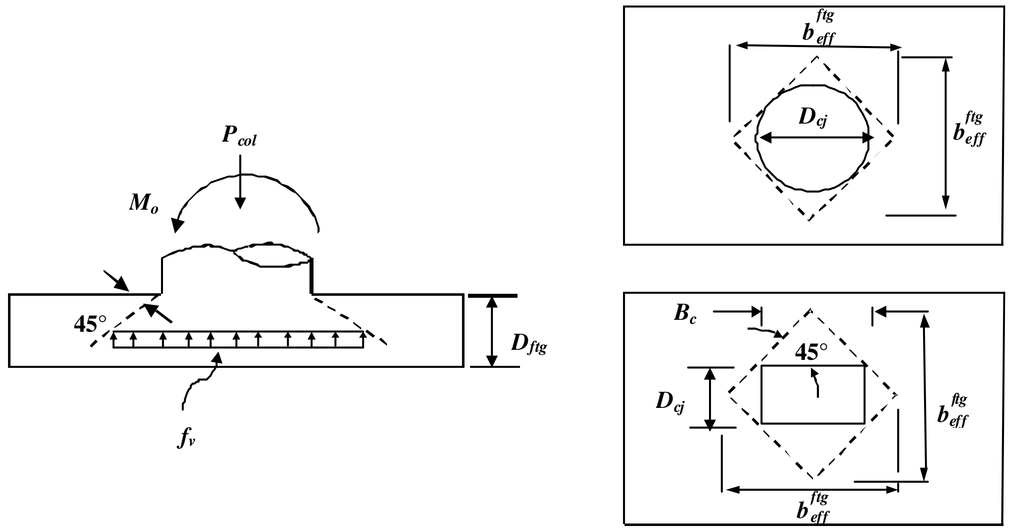

Figure 6.4.5-1—Effective Joint Width for Footing Joint Shear Stress

6.4.6—Effective Footing Width

For footings in SDCs C and D exhibiting rigid response and satisfying joint shear criteria, the entire width of the footing may be considered effective in resisting the column overstrength flexure and the associated shear when calculating the nominal section capacity. Otherwise, the effective footing width specified in Eq. 6.3.6-2 should be used.

6.4.7—Footing Joint Shear Reinforcement for SDCs C and D

Joint shear reinforcement shall be provided for all footings in SDC C and D. Where column moment continuity (fixity) is provided into the footing, horizontal joint reinforcement shall be provided by extending the column plastic hinge zone reinforcement into the footing to the point of tangency of the column bar hooks. The spacing of this reinforcement shall be the same as that in the adjacent plastic hinge zone. This reinforcement may be discontinuous at the top reinforcement layer of the footing. Figure 6.4.7-1 provides details of the reinforcement.

Additionally, vertical shear reinforcement (stirrups) shall be provided within a horizontal dimension from the face of column equal to the footing depth, D_{ftg}. The reinforcement shall be at least equivalent to #5 bars spaced at 12 in. each way and shall extend all the way around the column perimeter. Other failure modes that govern stirrup design may control over this prescriptive amount and shall be checked.

Where the joint principal tension stress calculated by Eq. 6.4.5-3 is less than $0.11\sqrt{f'_c}$, the stirrups may be terminated with a 180° hook on the top and a 90° hook on the bottom. Where the joint principal stress exceeds $0.11\sqrt{f'_c}$, the bottom of the stirrups shall terminate with either a mechanical head capable of developing the expected tensile strength of the stirrup or a 180° hook. If the latter is used, a full lap splice of the stirrup may be used

C6.4.7

The practice of turning column bars outward in footings to provide a stable base for construction of the column produces a somewhat undesirable joint shear force transfer mechanism. If the column bars are turned inward, the hooks on the bars are able to provide compression strut anchorages in the joint region. However, turning all the column bar hooks inward is generally not feasible. When the bars are turned outward, a more complex mechanism of force transfer involving the footing adjacent to the footing-column joint results. A detailed discussion of this phenomenon may be found in Priestley et al. (1996).

The provisions for reinforcement and detailing included in this Article provide minimum joint shear strength for resisting the column plastic hinging forces in the footing. The distribution of stirrups around the column provides for the joint shear forces that are required to be resisted just outside the joint area beneath the column. In most cases, no additional longitudinal steel is required to resist the inclined struts that accompany the shear-force transfer. In spread footings, the top steel is not fully utilized and is available for such use near the column. In pile caps, if the top steel is fully utilized, additional steel may be warranted for the joint shear mechanism. Design of such steel may follow the methods outlined in Priestley et al. (1996).

to facilitate construction. This restriction concerning the development of the bottom of the stirrup shall apply in a zone equal to half the width of the column in the direction under consideration. Beyond this zone, 90° hooks may be used on the bottom of the stirrups. These requirements are shown in Figure 6.4.7-1.

Where moment-reducing details are used to produce a pinned-base column, the extension of the column transverse steel may be omitted. The hinge detail may require its own transverse steel, and such steel is not affected by this requirement. Additionally, stirrups shall be placed around the perimeter of the column over the same width as for fixed-base columns. However, stirrups with 180° hooks on one end and 90° hooks on the other may be used to meet this requirement.

A critical aspect of the detailing is full development of the stirrups near the column. Consequently, positive development via mechanical heads or the use of 180° hooks with a lap splice in between is required when the calculated joint shear stress is over the nominal cracking principal tension stress, $0.11\sqrt{f'_c}$. This requirement is relaxed for stirrups further than $D_c/2$ from the column face. The permitted use of lap splices in the stirrups is because footings are capacity protected members and thus are not expected to experience damage that would compromise the efficacy of stirrup splices.

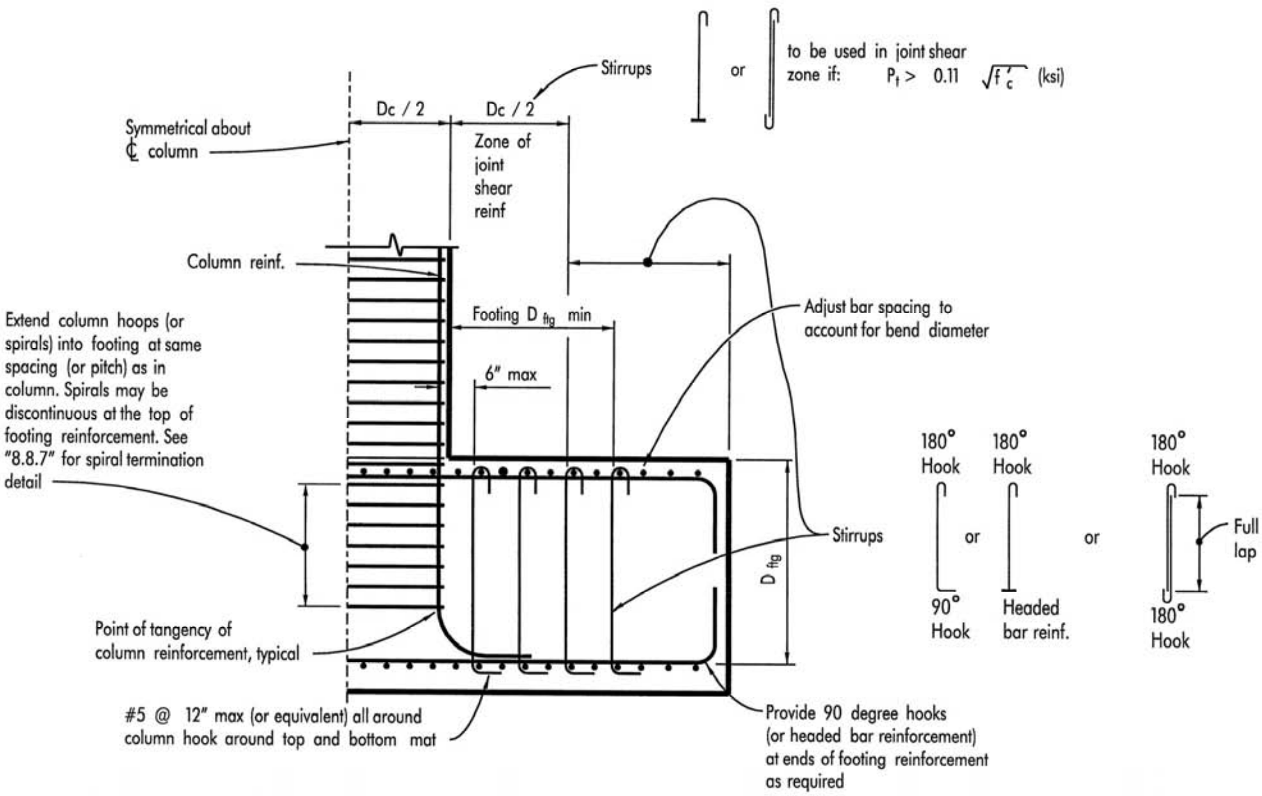

Figure 6.4.7-1—Footing Reinforcement—Fixed Column

6.5—DRILLED SHAFTS

Design requirements of drilled shafts shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinge locations shall be considered for SDCs B, C, and D.

The effects of liquefaction on loss of *P-y* strength shall be considered for locations where a potential for liquefaction occurs following the requirements in Article 6.8.

A stable length shall be ensured for a single column/shaft. The stable length shall be determined in accordance with Article 10.7.3.12 of the *AASHTO LRFD Bridge Design Specifications*, except that a load factor of 1.0 should be applied to the calculated lateral loads for the

C6.5

Various studies (Lam et al., 1998) have found that conventional *P-y* stiffnesses derived for driven piles are too soft for drilled shafts. This stiffer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 0.61 m (2 ft) is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the *P-y* curves. This adjustment is dependent on the construction method.

foundation. Overstrength properties may be used for the foundation and column elements.

The ultimate geotechnical capacity of single column/shaft foundation in compression and uplift shall not be exceeded under maximum seismic loads.

Base shear can also provide significant resistance to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the base of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases, the base conditions result in low interface strengths. For this reason, the amount of base shear to incorporate in lateral analyses will vary from case-to-case.

Lam et al. (1998) provides a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts. In the single-shaft configuration, the relative importance of axial and lateral response changes relative to, for example, a group of driven piles. Without the equivalent of a pile cap, lateral-load displacement of the shaft becomes more critical than the load-displacement relationships discussed above for driven piles.

The depth for stable conditions will depend on the stiffness of the rock or soil. Lower stable lengths are acceptable if the embedment length and the strength of drilled shaft provide sufficient lateral stiffness with adequate allowances for uncertainties in soil stiffness. In Caltrans' practice, a stability factor of 1.2 is applied to single-column bents supported on a pile shaft.

6.6—PILE EXTENSIONS

Design requirements of pile extensions shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinges locations shall be considered in SDCs B, C, and D.

The effects of liquefaction on loss of soil stiffness strength shall be considered in SDC B, C, and D. Group reduction factors shall be included in the analysis and design of pile extensions subjected to lateral loading in the transverse direction.

6.7—ABUTMENT DESIGN REQUIREMENTS

The participation of abutment walls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges in accordance with Article 5.2.

Abutment design shall be consistent with the demand model for the ERS used to assess intermediate substructure elements.

For conventional semi-gravity cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is considered acceptable. However, rotational instability may lead to collapse and thus shall be prevented.

6.7.1—Longitudinal Direction Requirements

The seismic design of free-standing abutments should take into account forces arising from seismically induced lateral earth pressures, additional forces arising from wall inertia effects, and the transfer of seismic forces from the bridge deck through bearing supports that do not slide freely (e.g., elastomeric bearings).

For free-standing abutments that may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the design approach shall be similar to that of a free-standing retaining wall, except that longitudinal force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outward from the wall.

Earthquake-induced active earth pressures should be computed using a horizontal acceleration of not less than 50 percent of the acceleration coefficient, A_s, unless supported by displacement analyses and approved by the Owner. The pseudostatic Mononobe–Okabe method of analysis should be used for computing lateral active soil pressures during seismic loading. The effects of vertical acceleration may be omitted.

Abutment displacements having a maximum drift of four percent of the wall height may be tolerated. A limiting equilibrium condition should be checked in the horizontal direction. If necessary, wall design (initially based on a static service loading condition) should be modified to meet the above condition.

For monolithic abutments in which the abutment forms an integral part of the bridge superstructure, the abutment shall be designed using one of the two alternatives depending on the contribution level accounted for in the analytical model as discussed in Article 5.2:

- *Case 1: Earthquake Resisting System (ERS) without Abutment Contribution.* At a minimum, the abutment shall be designed to resist the active pressure applied by the abutment backfill.
- *Case 2: Earthquake Resisting System (ERS) with Abutment Contribution.* If the abutment is part of the ERS and required to mobilize soil resistance, the full passive pressure may be used in developing the bridge model and should be used to design the end diaphragm.

For freestanding abutments that are restrained from horizontal displacement by anchors or concrete batter piles, earthquake-induced active earth pressures should be computed using a horizontal acceleration equal to the

C6.7.1

These Guide Specifications have been prepared to acknowledge the abutment to be used as an ERE and be a part of the ERS. If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

Use of the 50 percent reduction in A_s in the determination of seismic active earth pressure assumes that several inches of permanent movement of the wall will be permissible. The form of this movement will likely be a combination of sliding with some rotation. The potential consequences of this movement need to be considered when using the 50 percent reduction in A_s as a basis for the Mononobe–Okabe calculation.

acceleration coefficient, A_s, as a first approximation. The Mononobe–Okabe analysis method may be used. Up to a 50 percent reduction in the horizontal acceleration may be used, provided that the various components of the restrained wall can accommodate the increased level of displacement demand.

6.7.2—Transverse Direction Requirements

The provisions outlined in Article 5.2.4 shall be followed depending on the mechanism of transfer of superstructure transverse inertial forces to the bridge abutments and following the abutment contribution to the ERS applicable for SDCs C and D. These provisions should be considered for SDC B.

6.7.3—Other Requirements for Abutments

To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction should be considered for bridges less than 500 ft.

Settlement or approach slabs providing structural support between approach fills and abutments are recommended for all bridges in SDC D. Slabs shall be adequately linked to abutments using flexible ties.

For SDC D, the abutment skew should be minimized. The tendency for increased displacements at the acute corner of bridges with skewed abutments above 20° should be considered. In the case in which a large skew cannot be avoided, sufficient support length in conjunction with an adequate shear key shall be designed to ensure against any possible unseating of the bridge superstructure.

C6.7.3

During strong ground shaking such as will occur for SDC D, loose to medium dense soils making up the approach fill or located below the approach fill can densify. This densification will result in settlement of the roadway surface. The amount of settlement can range from negligible to a foot or more, particularly if layers of saturated sands or silts liquefy. This potential for settlement should be established during the geotechnical investigation for the site.

The differential settlement between a pile-supported abutment and the approach fill can result in a serious safety issue. Therefore, the recommended practice is to construct an approach slab that will provide a smooth transition between the abutment and the approach fill.

6.8—LIQUEFACTION DESIGN REQUIREMENTS

A liquefaction assessment shall be conducted for SDC C and D if both of the following conditions are present:

- *Groundwater Level*: The groundwater level anticipated at the site is within 50 ft of the existing ground surface or the final ground surface, whichever is lower.
- *Soil Characteristics*: Low plasticity silts and sands within the upper 75 ft are characterized by one of the following conditions: (1) the corrected standard penetration test (*SPT*) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and nonplastic silt layers, (2) the corrected cone penetration test (*CPT*) tip resistance, q_{ciN}, is less than or equal to 150 in sand and in non-plastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660 fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

C6.8

All of the following general conditions are necessary for liquefaction to occur:

- A sustained ground acceleration that is large enough and acting over a long enough period of time to develop excess pore-water pressure, thereby reducing effective stress and soil strength.
- Predominantly cohesionless soil that has the right gradation and composition. Liquefaction has occurred in soils ranging from low plasticity silts to gravels. Clean or silty sands and nonplastic silts are most susceptible to liquefaction.
- The state of the soil is characterized by a density that is low enough for the soil to exhibit contractive behavior when sheared undrained under the initial effective overburden stress.

Where loose to very loose saturated sands are within the subsurface soil profile such that liquefaction of these soils could impact the stability of the structure, the potential for liquefaction in SDC B should also be considered as discussed in the commentary.

For sites that require an assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction:

- Loss in strength in the liquefied layer or layers,
- Liquefaction-induced ground settlement, and
- Flow failures, lateral spreading, and slope instability.

For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows:

- *Nonliquefied Configuration*: The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a nonliquefied state.
- *Liquefied Configuration*: The structure as designed in nonliquefied configuration above shall be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified *P-y* curves, modulus of subgrade reaction, or *t-z* curves). The design spectrum shall be the same as that used in a nonliquefied configuration.

With the Owner's approval, or as required by the Owner, a site-specific response spectrum that accounts for the modifications in spectral content from the liquefying soil may be developed. Unless approved otherwise by the Owner, the reduced response spectrum resulting from the site-specific analyses shall not be less than two-thirds of the spectrum at the ground surface developed using the general procedure described in Article 3.4.1 modified by the site coefficients in Article 3.4.2.3.

The Designer should provide explicit detailing of plastic hinging zones for both cases mentioned above since it is likely that locations of plastic hinges for the liquefied configuration are different than locations of plastic hinges for the nonliquefied configuration. Design requirements including shear reinforcement should be met for the liquefied and nonliquefied configuration. Where liquefaction is identified, plastic hinging in the foundation may be permitted with the Owner's approval provided that the provisions of Article 3.3 are satisfied.

For those sites where liquefaction-related permanent lateral ground displacements (e.g., flow, lateral spreading, or slope instability) are determined to occur, the effects of lateral displacements on the bridge and retaining structures should be evaluated. These effects can include increased lateral pressure on bridge foundations and retaining walls.

- The presence of groundwater, resulting in a saturated or nearly saturated soil.

Methods used to assess the potential for liquefaction range from empirically-based design methods to complex numerical, effective stress methods that can model the time-dependent generation of pore-water pressure and its effect on soil strength and deformation. Furthermore, dynamic performance soil tests such as cyclic simple shear or cyclic triaxial tests can be used to assess liquefaction susceptibility and behavior to be used as input for liquefaction analysis and design.

The most common method of assessing liquefaction involves the use of empirical methods (e.g., Youd et al., 2001). These methods provide an estimate of liquefaction potential based on *SPT* blowcounts, *CPT* cone tip resistance, or shear wave velocity. This type of analysis should be conducted as a baseline evaluation, even when more rigorous methods are used.

Youd et al. (2001) summarizes the consensus of the profession up to year 2000 regarding the use of the simplified methods. Since the publication of this consensus paper, various other modifications to the consensus approach have been introduced, including those by Cetin et al. (2004), Moss et al. (2006), and Boulanger and Idriss (2006). These more recent methods account for additions to the database on liquefaction, as well as refinements in the interpretation of case history data. The newer methods potentially offer improved estimates of liquefaction potential and can be considered for use.

The simplified empirical methods are suited for use to a maximum depth of approximately 75 ft. This depth limit relates to the database upon which the original empirical method was developed. Most of the database was from observations of liquefaction at depths less than 50 to 60 ft. Extrapolation of the simplified method beyond 75 ft is therefore of uncertain validity. This limitation should not be interpreted as meaning liquefaction does not occur beyond 75 ft. Rather, different methods should be used for greater depths, including the use of site-specific ground motion response modeling in combination with liquefaction testing in the laboratory.

The magnitude for the design earthquake must be determined when conducting liquefaction assessments using the simplified empirical procedures. The earthquake magnitude used to assess liquefaction can be determined from earthquake deaggregation data for the site, available through the USGS national seismic hazard website http://earthquake.usgs.gov/research/hazmaps/ based on the 975-yr return period, i.e., five percent in 50 yr within the USGS website. If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the mean of the few dominant earthquakes in the deaggregation should be used.

Liquefaction is generally limited to granular soils, such as sands and nonplastic silts. Loose gravels also can liquefy if drainage is prevented such as might occur if a layer of clay or frozen soil is located over the gravel. Methods for eliminating sites based on soil type have

The effects of liquefaction-related, permanent lateral ground displacements on bridge and retaining wall performance should be considered separate from the inertial evaluation of the bridge structures. However, if large magnitude earthquakes dominate the seismic hazards, the bridge response evaluation should consider the potential simultaneous occurrence of:

- Inertial response of the bridge, and loss in ground response from liquefaction around the bridge foundations, and
- Predicted amounts of permanent lateral displacement of the soil.

been developed, as discussed by Youd et al, (2001), Bray and Sancio (2006), and Boulanger and Idriss (2006). These methods can be used to screen the potential for liquefaction in certain soil types. In the past soil screening with regard to silts was done using the Chinese criteria (Kramer, 1996). Recent studies (Bray and Sancio, 2006; Boulanger and Idriss, 2006) indicate that the Chinese criteria are unconservative, and therefore their use should be discontinued.

Two criteria for assessing liquefaction susceptibility of soils have been recently proposed as replacements to the Chinese criteria:

- Boulanger and Idriss (2006) recommend considering a soil to have clay-like behavior (i.e., not susceptible to liquefaction) if the plasticity index (PI) ħ 7.
- Bray and Sancio (2006) suggest that a soil with a $PI < 12$ and a ratio of water content to liquid limit (wc/LL) > 0.85 will be susceptible to liquefaction.

There is no current consensus on the preferred of the two criteria, and, therefore, either method may be used, unless the Owner has a specific preference.

To determine the location of soils that are adequately saturated for liquefaction to occur, the seasonally averaged groundwater elevation should be used. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of fluctuation.

Liquefaction evaluation is required only for sites meeting requirements for SDC C and D, provided that the soil is saturated and of a type that is susceptible to liquefaction. For loose to very loose sand sites (e.g., $(N_1)_{60} < 10$ bpf or $q_{c1N} < 75$), a potential exists for liquefaction in SDC B, if the acceleration coefficient, A_s, is 0.15 or higher. The potential for and consequences of liquefaction for these sites will depend on the dominant magnitude for the seismic hazard. As the magnitude decreases, the liquefaction resistance of the soil increases due to the limited number of earthquake loading cycles. Generally, if the magnitude is 6 or less, even in these very loose soils, either the potential for liquefaction is very low or the extent of liquefaction is very limited. Nevertheless, a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and A_s is greater than or equal to 0.15. These loose to very loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts.

During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement include: