7.2　Foundation Modeling for Seismic Loads

7.2.1　General

Bridge modeling for seismic events shall be in accordance with requirements of the AASHTO Seismic Section 5, "Analytical Models and Procedures." The following guidance is for elastic dynamic analysis. Refer to AASHTO Seismic 5.4 for other dynamic analysis procedures.

~~The following sections were originally developed for a force-based seismic design as required in previous versions of the AASHTO LRFD Specifications. Modifications have been made to the following sections to incorporate the provisions of the new AASHTO Seismic Specifications. It is anticipated that this section will continue to be revised as more experience is gained through the application of the AASHTO Seismic Specifications.~~

7.2.2　Substructure Elastic Dynamic Analysis Procedure

The following is a general description of the iterative process used in an elastic dynamic analysis. *Note:* An elastic dynamic analysis is needed to determine the displacement demand, Δ_D. The substructure elements are first designed using Strength, Service or Extreme II limit state load cases prior to performing the dynamic analysis.

1. Build a Finite Element Model (FEM) to determine initial structure response (*EQ+DL*). Assume that foundation springs are located at the bottom of the column.

    A good initial assumption for fixity conditions of deep foundations (shafts or piles) is to add 10′ to the column length in stiff soils and 15′ to the column in soft soils.

    Use multi-mode response spectrum analysis to generate initial displacements.

2. Determine foundation springs using results from the seismic analysis in the longitudinal and transverse directions. *Note:* The load combinations specified in AASHTO Seismic 4.4 shall NOT be used in this analysis.

3. For spread footing foundations, the FEM will include foundation springs calculated based on the footing size as calculated in Section 7.2.7 of this manual. No iteration is required unless the footing size changes. *Note:* For Site Classes A and B the AASHTO Seismic Specification allows spread footings to be modeled as rigid or fixed.

4. For deep foundation analysis, the FEM and the soil response program must agree or converge on soil/structure lateral response. In other words, the moment, shear, deflection, and rotation of the two programs should be within 10 percent. More iteration will provide convergence much less than 1 percent. The iteration process to converge is as follows:

    a. Apply the initial FEM loads (moment and shear) to a p-y type soil response program such as ~~DFSAP~~ ~~the Ensoft applications~~ LPILE (including LPILE, LPILE-SHAFT and LPILE-GROUP) ~~L-Pile, L-Pile Sgaftand L-Pile Group.~~ ~~DFSAP is a program that models Short, Intermediate or Long shafts or piles using the Strain Wedge Theory. See discussion below for options and applicability of DFSAP and Lpile soil response programs.~~

    b. Calculate foundation spring values for the FEM. *Note:* The load combinations specified in AASHTO Seismic 4.4 shall not be used to determine foundation springs.

    c. Re-run the seismic analysis using the foundation springs calculated from the soil response program. The structural response will change. Check to insure the FEM results (M, V, Δ, θ, and spring values) in the transverse and longitudinal direction are within 10 percent of the previous run. This check verifies the linear spring, or soil response (calculated by the FEM) is close to the predicted

nonlinear soil behavior (calculated by the soil response program). If the results of the FEM and the soil response program differ by more than 10 percent, recalculate springs and repeat steps (a) thru (c) until the two programs converge to within 10 percent.

Special note for single column/single shaft configuration: The seismic design philosophy requires a plastic hinge in the substructure elements above ground (preferably in the columns). Designers should note the magnitude of shear and moment at the top of the shaft, if the column "zero" moment is close to a shaft head foundation spring, the FEM and soil response program will not converge and plastic hinging might be below grade.

Throughout the iteration process it is important to note that any set of springs developed are only applicable to the loading that was used to develop them (due to the inelastic behavior of the soil in the foundation program). This can be a problem when the forces used to develop the springs are from a seismic analysis that combines modal forces using a method such as the Complete Quadratic Combination (CQC) or other method. The forces that result from this combination are typically dominated by a single mode (in each direction as shown by mass participation). This results in the development of springs and forces that are relatively accurate for that structure. If the force combination (CQC or otherwise) is not dominated by one mode shape (in the same direction), the springs and forces that are developed during the above iteration process may not be accurate.

~~Guidelines for the use of DFSAP and Lpile programs:~~

- ~~The DFSAP Program may be used for pile and shaft foundations for static soil structural analysis cases.~~

- ~~The DFSAP Program may be used for pile and shaft foundations for liquefied soil structural analysis case of a shaft or pile foundation with static soil properties reduced by the Geotechnical Branch to account for effects of liquefaction. The Liquefaction option in either Lpile or DFSAP programs shall not be used (the liquefaction option shall be disabled). The Liquefied Sand soil type shall not be used in Lpile~~LPILE may be used for a pile group-supported footing. Pile or shaft foundation group effects for lateral loading shall be taken as recommended in the project geotechnical report. The Liquefaction option in LPILE shall not be used (the liquefaction option shall be disabled). The "Liquefied Sand" soil type shall not be used in LPILE.

- ~~The Lpile Program may be used for a pile supported foundation group. Pile or shaft foundation group effect efficiency shall be taken as recommended in the project geotechnical report.~~

7.2.3 Bridge Model Section Properties

In general, gross section properties may be assumed for all FEM members, except concrete columns.

A. **Cracked Properties for Columns –** Effective section properties shall be in accordance with the AASHTO Seismic Section 5.6.

B. **Shaft Properties –** The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows.

For a stiff substructure response:

1. Use 1.5 f'_c to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.

2. Use I_g based on the maximum oversized shaft diameter allowed by Section 6-19 of the *Standard Specifications*.

3. When permanent casing is specified, increase shaft I_g using the transformed area of a ¾″ thick casing. Since the contractor will determine the thickness of the casing, ¾″ is a conservative estimate for design.

For a soft substructure response:

1. Use $0.85\,f'_c$ to calculate the modulus of elasticity. Since the quality of shaft concrete can be suspect when placed in water, the factor of 0.85 is an estimate for a decrease in stiffness.

2. Use I_g based on the nominal shaft diameter. Alternatively, I_e may be used when it is reflective of the actual load effects in the shaft.

3. When permanent casing is specified, increase I using the transformed area of a ⅜″ thick casing.

   Since the contractor will determine the thickness of the casing, ⅜″ is a minimum estimated thickness for design.

C. **Cast-in-Place Pile Properties** – For a stiff substructure response:

1. Use $1.5\,f'_c$ to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a ̶reasonable estimate for an increase in stiffness.

2. Use the pile I_g plus the transformed casing moment of inertia.
   ~~*Note:* If DFSAP is used for analysis, the reinforcing and shell properties are input and the moment of inertia is computed internally.~~

	(7.2.3-1)
Where: n = E_s/E_c	

Use a steel casing thickness of ¼″ for piles less than 14″ in diameter, ⅜″ for piles 14″ to 18″ in diameter, and ½″ for larger piles.
Note: These casing thicknesses are to be used for analysis only, the contractor is responsible for selecting the casing thickness required to drive the piles.

For a soft substructure response:

1. Use $1.0\,f'_c$ to calculate the modulus of elasticity.

2. Use pile I_g, neglecting casing properties.

7.2.4 Bridge Model Verification

As with any FEM, the designer should review the foundation behavior to ensure the foundation springs correctly imitate the known boundary conditions and soil properties. Watch out for mismatch of units.

All finite element models must have dead load static reactions verified and boundary conditions checked for errors. The static dead loads (*DL*) must be compared with hand calculations or another program's results. For example, span member end moment at the supports can be released at the piers to determine simple span reactions. Then hand calculated simple span *DL* or PGsuper *DL* and *LL* is used to verify the model.

Crossbeam behavior must be checked to ensure the superstructure *DL* is correctly distributing

to substructure elements. A 3D bridge line model concentrates the superstructure mass and stresses to a point in the crossbeam. Generally, interior columns will have a much higher loading than the exterior columns. To improve the model, crossbeam I_g should be increased to provide the statically correct column DL reactions. This may require increasing I_g by about 1000 times. Many times this is not visible graphically and should be verified by checking numerical output. Note that most finite element programs have the capability of assigning constraints to the crossbeam and superstructure to eliminate the need for increasing the I_g of the crossbeam.

Seismic analysis may also be verified by hand calculations. Hand calculated fundamental mode shape reactions will be approximate; but will ensure design forces are of the same magnitude.

Designers should note that additional mass might have to be added to the bridge FEM for seismic analysis. For example, traffic barrier mass and crossbeam mass beyond the last column at piers may contribute significant weight to a two-lane or ramp structure.

7.2.5   Deep Foundation Modeling Methods

A designer must assume a foundation support condition that best represents the foundation behavior. Deep foundation elements attempt to imitate the non-linear lateral behavior of several soil layers interacting with the deep foundation. The bridge FEM then uses the stiffness of the element to predict the seismic structural response. Models using linear elements that are not based on non-linear soil-structure interaction are generally considered inaccurate for soil response/element stress and are not acceptable. There are three methods used to model deep foundations (FHWA Report No. 1P-87-6). Of these three methods the Bridge and Structures Office prefers Method II for the majority of bridges.

A. **Method I – Equivalent Cantilever Column –** This method assumes a point of fixity some depth below the bottom of the column to model the stiffness of the foundation element. This shall only be used for a preliminary model of the substructure response in SDC C and D.

B. **Method II – Equivalent Base Springs –** This method models deep foundations by using a {6x6} matrix. There are two techniques used to generate the stiffness coefficients for the foundation matrix. The equivalent stiffness coefficients assessed are valid only at the given level of loading. Any changes of the shaft-head loads or conditions will require a new run for the program to determine the new values of the equivalent stiffness coefficients. These equivalent stiffness coefficients account for the nonlinear response of shaft materials and soil resistance.

    Technique I – The matrix is generated, using superposition, to reproduce the non-linear behavior of the soil and foundation at the maximum loading. With Technique I, 10 terms are produced, 4 of these terms are "cross couples." Soil response programs~~, such as Lpile or DFSAP,~~ analyze the non-linear soil response. The results are then used to determine the equivalent base springs. See Appendix 7-B1 for more information.

    Technique II – The equivalent stiffness matrix generated using this technique uses only the diagonal elements (no cross coupling stiffnesses). ~~The DFSAP program shall be used to develop the equivalent stiffness matrix.~~ This technique is recommended to construct the foundation stiffness matrix (equivalent base springs).

    In Technique II the "cross couple" effects are internally accounted for as each stiffness element and displacement is a function of the given Lateral load (P) and Moment (M). Technique II uses the total response ($\Delta_{t(P,M)}$ $\theta_{t(P,M)}$) to determine displacement and equivalent soil stiffness, maintaining a nonlinear analysis. Technique I requires superposition by adding the individual responses due to the lateral load and moment to determine displacement and soil stiffness. Using superposition to combine two nonlinear responses results in errors in displacement and stiffness for the total response as seen in the Figure 7.2.5-1. As illustrated, the total response due to lateral load (P) and moment (M) does not

necessarily equal the sum of the individual responses. ~~For more details on the equivalent stiffness matrix, see the DFSAP reference manual.~~

C. **Method III – Non-Linear Soil Springs** – This method attaches non-linear springs along the length of deep foundation members in a FEM model. See Appendix 7-B2 for more information. This method has the advantage of solving the superstructure and substructure seismic response simultaneously. The soil springs must be nonlinear PY curves and represent the soil/structure interaction. This cannot be done during response spectrum analysis with some FEM programs.

D. **Spring Location (Method II)** – The preferred location for a foundation spring is at the bottom of the column. This includes the column mass in the seismic analysis. For design, the column forces are provided by the FEM and the soil response program provides the foundation forces. Springs may be located at the top of the column. However, the seismic analysis will not include the mass of the columns. The advantage of this location is the soil/structure analysis includes both the column and foundation design forces.

$$y_{P,M} > y_P + y_M$$

Limitations on the Technique I (Superposition Technique)
Figure 7.2.5-1
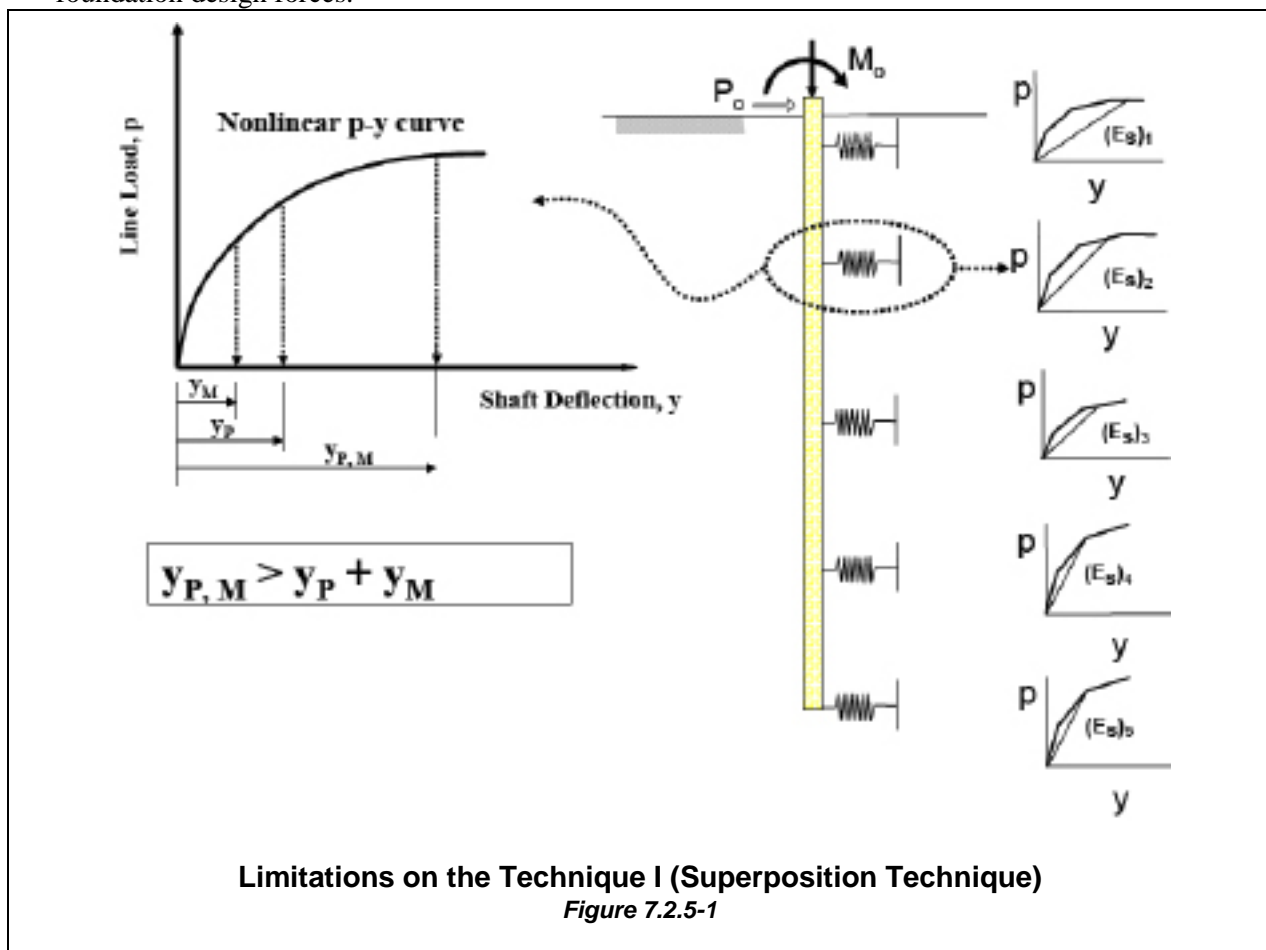
Designers should be careful to match the geometry of the FEM and soil response program. If the location of the foundation springs (or node) in the FEM does not match the location input to the soil response program, the two programs will not converge correctly.

E. **Boundary Conditions (Method II)** – To calculate spring coefficients, the designer must first identify the predicted shape, or direction of loading, of the foundation member where the spring is located in the bridge model. This will determine if one or a combination of two boundary conditions apply for the

transverse and longitudinal directions of a support.

A fixed head boundary condition occurs when the foundation element is in double curvature where translation without rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the opposite direction of applied moment. This is a common assumption applied to both directions of a rectangular pile group in a pile supported footing.

A free head boundary condition is when the foundation element is in single curvature where translation and rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the same direction as the applied moment. Most large diameter shaft designs will have a single curvature below ground line and require a free head assumption. The classic example of single curvature is a single column on a single shaft. In the transverse direction, this will act like a flagpole in the wind, or free head. What is not so obvious is the same shaft will also have single curvature in the longitudinal direction (below the ground line), even though the column exhibits some double curvature behavior. Likewise, in the transverse direction of multi-column piers, the columns will have double curvature (frame action). The shafts will generally have single curvature below grade and the free head boundary condition applies. The boundary condition for large shafts with springs placed at the ground line will be free head in most cases.

The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams for the structure. If the sign convention is still unclear, apply a unit load in a separate static FEM run to establish sign convention at the point of interest.

The correct boundary condition is critical to the seismic response analysis. For any type of soil and a given foundation loading, a fixed boundary condition will generally provide soil springs four to five times stiffer than a free head boundary condition.

F. **Spring Calculation (Method II)** – The first step to calculate a foundation spring is to determine the shear and moment in the structural member where the spring is to be applied in the FEM. Foundation spring coefficients should be based on the maximum shear and moment from the applied longitudinal OR transverse seismic loading. The combined load case ($1.0L$ and $0.3T$) shall be assumed for the design of structural members, and NOT applied to determine foundation response. For the simple case of a bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat unclear for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

G. **Matrix Coordinate Systems (Method II)** – The Global coordinate systems used to demonstrate matrix theory are usually similar to the system defined for substructure loads in Section 7.1.3 of this manual, and is shown in Figure 7.2.5-2. This is also the default Global coordinate system of GTS~~trudl~~TRUDL. This coordinate system applies to this Section to establish the sign convention for matrix terms. Note vertical axial load is labeled as P, and horizontal shear load is labeled as V.

Also note the default Global coordinate system in ~~SAP 2000~~ CSIBRIDGE uses Z as the vertical axis (gravity axis). When imputing spring values in ~~SAP2000~~CSIBRIDGE the coefficients in the stiffness matrix will need to be adjusted accordingly. ~~SAP2000~~CSIBRIDGE allows you to assign spring stiffness values to support joints. By default, only the diagonal terms of the stiffness matrix can be assigned, but when selecting the advanced option, terms to a symmetrical {6x6} matrix can be assigned.

┌───┐
│   │
│   **Global Coordinate System**   │
│   *Figure 7.2.5-2*   │
│   │
└───┘

H. **Matrix Coefficient Definitions (Method II) –** The stiffness matrix containing the spring values and using the standard coordinate system is shown in Figure 7.2.5-3. (Note that cross-couple terms generated using Technique I are omitted). For a description of the matrix generated using Technique I see Appendix 7-B1. The coefficients in the stiffness matrix are generally referred to using several different terms. Coefficients, spring or spring value are equivalent terms. Lateral springs are springs that resist lateral forces. Vertical springs resist vertical forces.

┌───┐
│   │
│   **Standard Global Matrix**   │
│   *Figure 7.2.5-3*   │
│   │
└───┘

Where the linear spring constants or K values are defined as follows, using the Global Coordinates:

K11 = Longitudinal Lateral Stiffness (kip/in)
K22 = Vertical or Axial Stiffness (kip/in)
K33 = Transverse Lateral Stiffness (kip/in)
K44 = Transverse Bending or Moment Stiffness (kip-in/rad)
K55 = Torsional Stiffness (kip-in/rad)
K66 = Longitudinal Bending or Moment Stiffness (kip-in/rad)

The linear lateral spring constants along the diagonal represent a point on a non-linear soil/structure response curve. The springs are only accurate for the applied loading and less accurate for other loadings. This is considered acceptable for Strength and Extreme Event design. ~~For calculation of spring constants for Technique I see Appendix 7-B1. For calculation of spring constants for Technique II see the DFSAP reference manual.~~

I. **Group Effects –** When a foundation analysis uses ~~Lpile~~LPILE or an analysis using PY relationships, group effects will require the geotechnical properties to be reduced before the spring values are calculated. The geotechnical report will provide transverse and longitudinal multipliers that are applied to the PY curves. This will reduce the pile resistance in a linear fashion. The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in the WSDOT *Geotechnical Design Manual* M 46-03, Section 8.12.2.5.

~~Group effect multipliers are not valid when the DFSAP program is used. Group effects are calculated internally using Strain Wedge Theory.~~

J. **Shaft Caps and Pile Footings –** Where pile supported footings or shaft caps are entirely below grade, their passive resistance should be utilized. In areas prone to scour or lateral spreading, their passive resistance should be neglected. ~~DFSAP has the capability to account for passive resistance of footings and caps below ground.~~

7.2.6 Lateral Analysis of Piles and Shafts

7.2.6.1 Determination of Tip Elevations

Lateral analysis of piles and shafts involves determination of a shaft or pile tip location sufficient to resist lateral loads in both orthogonal directions. In many cases, the shaft or pile tip depth required to resist lateral loads may be deeper than that required for bearing or uplift. However, a good starting point for a tip elevation is the depth required for bearing or uplift. Another good "rule-of-thumb" starting point for shaft tips is an embedment depth of 6 diameters ($6D$) to 8 diameters ($8D$). Refer also to the geotechnical report minimum tip elevations provided by the geotechnical engineer.

A parametric study or analysis should be performed to evaluate the sensitivity of the depth of the shaft or pile to the displacement of the structure (i.e. the displacement of the shaft or pile head) in order to determine the depth required for stable, proportionate lateral response of the structure. Determination of shaft or pile tip location requires engineering judgment, and consideration should be given to the type of soil, the confidence in the soil data (proximity of soil borings) and the potential variability in the soil profile. Arbitrarily deepening shaft or pile tips may be conservative but can also have significant impact on constructability and cost.

The following is a suggested approach for determining appropriate shaft or pile tip elevations that are located in soils. Other considerations will need to be considered when shaft or pile tips are located in rock, such as the strength of the rock. This approach is based on the displacement demand seismic design procedures specified in the AASHTO Seismic Specifications.

1. Size columns and determine column reinforcement requirements for Strength and Service load cases.

2. Determine the column plastic over-strength moment and shear at the base of the column using the axial dead load and expected column material properties. A program such as ~~Xtract~~XTRACT or ~~SAP2000~~CSIBRIDGE may be used to help compute these capacities. The plastic moments and shears are good initial loads to apply to a soil response program ~~(DFSAP or Lpile)~~. In some cases, Strength or other Extreme event loads may be a more appropriate load to apply in the lateral analysis. For example, in eastern Washington seismic demands are relatively low and elastic seismic or Strength demands may control.

3. Perform lateral analysis using the appropriate soil data from the Geotechnical report for the given shaft or pile location. If final soil data is not yet available, consult with the Geotechnical engineer for preliminary values to use for the site.

    Note: Early in the lateral analysis it is wise to obtain moment and shear demands in the shaft or pile and check that reasonable reinforcing ratios can be used to resist the demands. If not, consider resizing the foundation elements and restart the lateral analysis.

4. Develop a plot of embedment depth of shaft or pile versus lateral deflection of the top of shaft or pile. The minimum depth, or starting point, shall be the depth required for bearing or uplift or as specified by the geotechnical report. An example plot of an 8′ diameter shaft is shown in Figure 7.2.6-1 and illustrates the sensitivity of the lateral deflections versus embedment depth. Notice that at tip depths of approximately 50′ (roughly $6D$) the shaft head deflections begin to increase substantially with small reductions in embedment depth. The plot also clearly illustrates that tip embedment below 70′ has no impact on the shaft head lateral deflection.

5. From the plot of embedment depth versus lateral deflection, choose the appropriate tip elevation. In the example plot in Figure 7.2.6-1, the engineer should consider a tip elevation to the left of the dashed vertical line drawn in the Figure. The final tip elevation would depend on the confidence in the soil data and the tolerance of the structural design displacement. For example, if the site is prone to variability in soil layers, the engineer should consider deepening the tip; say 1 to 3 diameters, to ensure that

embedment into the desired soil layer is achieved. The tip elevation would also depend on the acceptable lateral displacement of the structure. To assess the potential variability in the soil layers, the geotechnical engineer assigned to the project should be consulted.

Shaft Head Deflection (in)

Shaft Tip Elevation vs Shaft Head Deflection
Figure 7.2.6-1
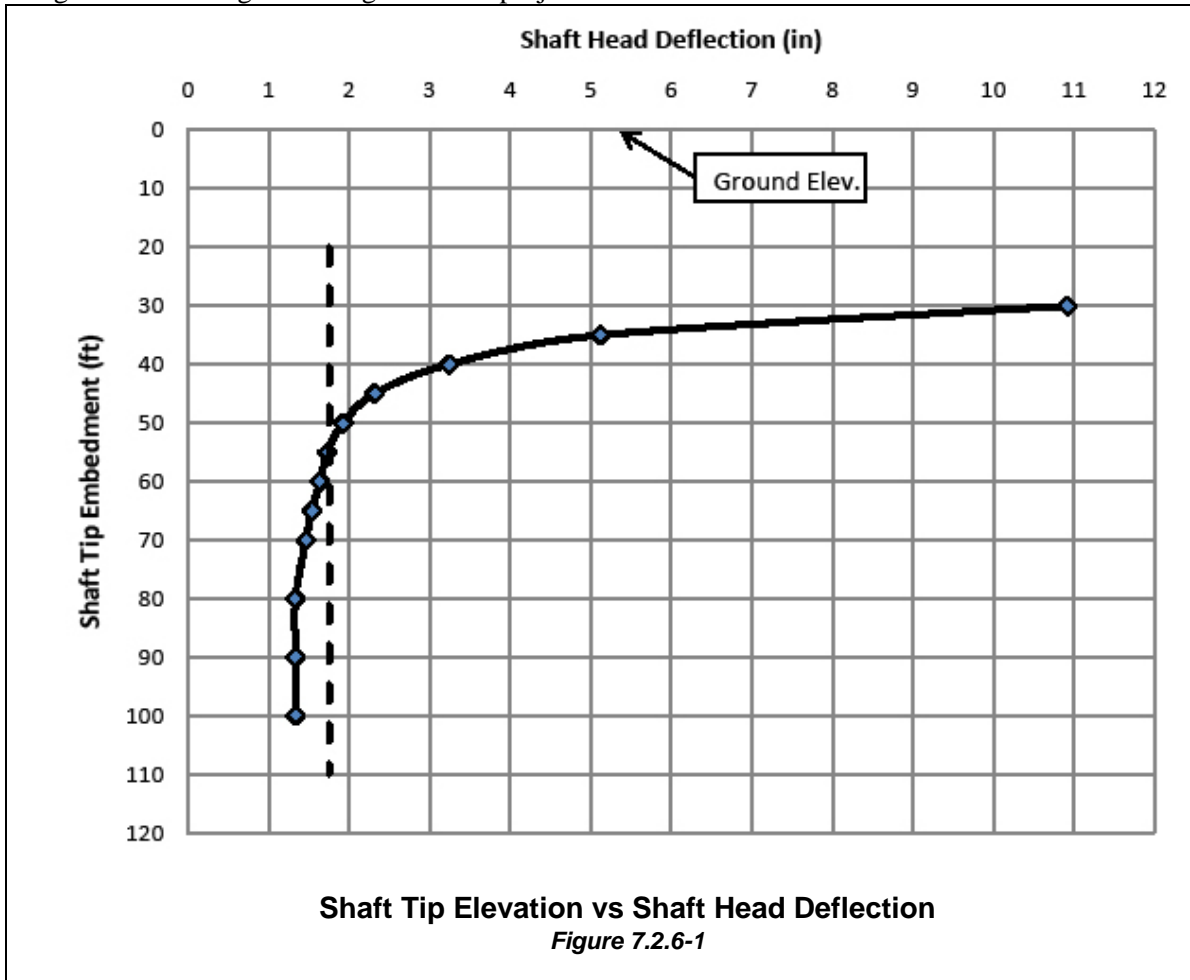
6. With the selected tip elevation, review the deflected shape of the shaft or pile, which can be plotted in ~~DFSAP or Lpile~~LPILE. Examples are shown in Figure 7.2.6-2. Depending on the size and stiffness of the shaft or pile and the soil properties, a variety of deflected shapes are possible, ranging from a rigid body (fence post) type shape to a long slender deflected shape with 2 or more inflection points. Review the tip deflections to ensure they are reasonable, particularly with rigid body type deflected shapes. Any of the shapes in the Figure may be acceptable, but again it will depend on the lateral deflection the structure can tolerate.

Various Shaft Deflected Shapes
Figure 7.2.6-2

The engineer will also need to consider whether liquefiable soils are present and/or if the shaft or pile is within a zone where significant scour can occur. In this case the analysis needs to be bracketed to envelope various scenarios. It is likely that a liquefiable or scour condition case may control deflection. In general,

the WSDOT policy is to not include scour with Extreme Event I load combinations. In other words, full seismic demands or the plastic over-strength moment and shear, are generally not applied to the shaft or pile in a scoured condition. However, in some cases a portion of the anticipated scour will need to be included with the Extreme Event I load combination limit states. When scour is considered with the Extreme Event I limit state, the soil resistance up to a maximum of 25 percent of the scour depth for the design flood event (100 year) shall be deducted from the lateral analysis of the pile or shaft. In all cases where scour conditions are anticipated at the bridge site or specific pier locations, the geotechnical engineer and the Hydraulics Branch shall be consulted to help determine if scour conditions should be included with Extreme Event I limit states.

If liquefaction can occur, the bridge shall be analyzed using both the static and liquefied soil conditions. The analysis using the liquefied soils would typically yield the maximum bridge deflections and will likely control the required tip elevation, whereas the static soil conditions may control for strength design of the shaft or pile.

Lateral spreading is a special case of liquefied soils, in which lateral movement of the soil occurs adjacent to a shaft or pile located on or near a slope. Refer to the WSDOT *Geotechnical Design Manual* M 46-03 for discussion on lateral spreading. Lateral loads will need to be applied to the shaft or pile to account for lateral movement of the soil. There is much debate as to the timing of the lateral movement of the soil and whether horizontal loads from lateral spread should be combined with maximum seismic inertia loads from the structure. Most coupled analyses are 2*D*, and do not take credit for lateral flow around shafts, which can be quite conservative. The AASHTO Seismic Spec. permits these loads to be uncoupled; however, the geotechnical engineer shall be consulted for recommendations on the magnitude and combination of loads. See WSDOT *Geotechnical Design Manual* M 46-03 Sections 6.4.2.8 and 6.5.4.2 for additional guidance on combining loads when lateral spreading can occur.

7.2.6.2   Pile and Shaft Design for Lateral Loads

The previous section provides guidelines for establishing tip elevations for shafts and piles. Sensitivity analyses that incorporate both foundation and superstructure kinematics are often required to identify the soil conditions and loadings that will control the tip, especially if liquefied or scoured soil conditions are present. Several conditions will also need to be analyzed when designing the reinforcement for shafts and piles to ensure the controlling case is identified. All applicable strength, service and extreme load cases shall be applied to each condition. A list of these conditions includes, but is not limited to the following:

1. Static soil properties with both stiff and soft shaft or pile properties. Refer to Sections 7.2.3(B) and 7.2.3(C) for guidelines on computing stiff and soft shaft or pile properties.

2. Dynamic or degraded soil properties with both stiff and soft shaft or pile properties.

3. Liquefied soil properties with both stiff and soft shaft or pile properties.

    a. When lateral spreading is possible, an additional loading condition will need to be analyzed. The geotechnical engineer shall be consulted for guidance on the magnitude of seismic load to be applied in conjunction with lateral spreading loads. See WSDOT *Geotechnical Design Manual* M 46-03 Sections 6.4.2.8 and 6.5.4.2 for additional guidance on combining loads when lateral spreading can occur.

4. Scour condition with stiff and soft shaft or pile properties. The scour condition is typically not combined with Extreme Event I load combinations, however the designer shall consult with the Hydraulics Branch and geotechnical engineer for recommendations on load combinations. If scour is considered with the Extreme Event I limit state, the analysis should be conducted assuming that the soil in the upper 25 percent of the estimated scour depth for the design (100 year) scour event has been removed to determine the available soil resistance for the analysis of the pile or shaft.

Note: Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and degradation and/or liquefaction of the soil tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections are low. Later in the earthquake, the soil-structure stiffness is lower and deflections higher. This phenomenon is normally addressed by bracketing the analyses as discussed above.

However, in some cases a site specific procedure may be required to develop a site specific design response spectrum. A site specific procedure may result in a reduced design response spectrum when compared to the general method specified in the AASHTO Seismic 3.4. Section 3.4 requires the use of spectral response parameters determined using USGA/AASHTO Seismic Hazard Maps. The AASHTO Seismic Spec. limits the reduced site specific response spectrum to two-thirds of what is produced using the general method. Refer to the *Geotechnical Design Manual* M 46-03 Chapter 6 for further discussion and consult the geotechnical engineer for guidance.

Refer to Section7.8 Shafts and Chapter 4 for additional guidance/requirements on design and detailing of shafts and Section 7.9 Piles and Piling and Chapter 4 for additional guidance/requirements on design and detailing of piles.

7.2.7 Spread Footing Modeling

For a first trial footing configuration, Strength column moments or column plastic hinging moments may be applied to generate footing dimensions. Soil spring constants are developed using the footing plan area, thickness, embedment depth, Poisson's ratio ν, and shear modulus G. The Geotechnical Branch will provide the appropriate Poisson's ratio and shear modulus. Spring constants for shallow rectangular footings are obtained using the following equations developed for rectangular footings. This method for calculating footing springs is referenced in ASCE 41-06, Section 4.4.2.1.2. (*Note:* ASCE 41-06 was developed from FEMA 356.)

Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.
Figure 7.2.7-1

Where:
K = βK_{sur}
K = Translation or rotational spring
K_{sur} = Stiffness of foundation at surface, see Table 7.2.7-1
β = Correction factor for embedment, see Table 7.2.7-2

Degree of Freedom	$\mathbf{K}_{sur}$
Translation along x-axis	
Translation along y-axis	
Translation along z-axis	
Rocking about x-axis	
Rocking about y-axis	

Torsion about z-axis	

Stiffness of Foundation at Surface
Table 7.2.7-1

Where:

d = Height of effective sidewall contact (may be less than total foundation height if the foundation is exposed).

h = Depth to centroid of effective sidewall contact.

Figure 7.2.7-2

Degree of Freedom	β
Translation along x-axis	
Translation along y-axis	
Translation along z-axis	
Rocking about x-axis	
Rocking about y-axis	
Torsion about z-axis	

Correction Factor for Embedment
Table 7.2.7-2