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From: John Lawson
Manager, Geotechnical Design Section
Materials Group (068R)

Subject: Geotechnical Design Policy DS-3
Load Resistance Factor Design (LRFD)
Analysis of Drilled Shafts Subjected to Lateral Loads based on Load and Resistance Factor Design (LRFD) Methodology

The AASHTO (2010) LRFD Bridge Design Specifications are mandatory for all federally funded projects. The purpose of this policy memorandum is to provide guidance for analysis of drilled shafts under lateral loads using LRFD methodology. Recommendations are presented for determining shaft length, minimum requirements for analytical models, depth to fixity and considerations for collapse-susceptible soils.

Personnel, both within ADOT and design consultants working on projects that require LRFD for substructures, shall follow the attached policy. The designer should contact the ADOT Materials Group for an updated version of this policy in the event any interim revisions are made to AASHTO (2010) or a new edition of AASHTO is issued.

If you have any questions regarding this design policy please contact Jim Wilson at 602-712-8081 or John Lawson at 602-712-8130.

Arizona Department of Transportation

Materials Group - Geotechnical Design Section

MEMORANDUM

To:	Date: December 1, 2010
John Lawson, P.E., Manager, Geotechnical Design Section **Jean Nehme, Ph.D., P.E., State Bridge Engineer**	

From:	Subject: Analysis of Drilled Shafts Subjected to Lateral Loads Based on Load and Resistance Factor (LRFD) Methodology[1]
Norman H. Wetz, P.E., Senior Geotechnical Engineer **James D. Wilson, P.E., Geotechnical Planning Engineer** **Amin Islam, Ph.D., P.E., Bridge Technical Section Leader** **Navaphan Viboolmate, P.E., Bridge Design Section Leader**	

ADOT POLICY MEMORANDUM: ADOT DS-3

This memorandum presents guidance for analysis of drilled shafts[2] under lateral loads for the following aspects:

I. Shaft Length
II. Minimum requirements for analytical models
III. Depth to fixity
IV. Considerations for collapse-susceptible soils

I. Shaft Length, L_L, Based on Lateral Load Analysis

While using Load and Resistance Factor Design (LRFD) methodology based on AASHTO (2010), the shaft length, L_L, is determined from considerations of the overturning strength limit state (geotechnical stability), structural strength limit state, and structural service limit state. The following procedure is recommended for determining the shaft length, L_L:

1. Select an appropriate soil-structure interaction analysis procedure as discussed in Section II of this memorandum.
2. Select a shaft diameter. Perform lateral load analysis using applicable strength limit state factored loads, nominal lateral geotechnical resistances (i.e., resistance factor = 1.0) computed in accordance with Article 10.7.3.12 of AASHTO (2010) and bending stiffness computed as $E_C I_G$, where E_C is the elastic modulus of concrete and I_G is the gross (uncracked) moment of inertia for the selected shaft diameter. Since the analysis is iterative, select an initial shaft length, L_{LONG}, that is obviously long, e.g., a shaft length corresponding to 10 to 15 times the shaft diameter.

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1 This memorandum is based on AASHTO (2010) – 5th Edition. The designer should contact ADOT Materials Group for an updated version of this memorandum in the event any interim revisions to AASHTO (2010) are issued or a new edition of AASHTO is issued. Additionally, the designer should ensure conformance with Section 10 (Foundations) of the latest ADOT Bridge Design Guidelines found at the following web link: http://www.azdot.gov/Highways/bridge/Guidelines/DesignGuidelines/index.asp.

2 Drilled shafts are primarily addressed in this memorandum. The recommendations are also generally applicable to driven piles or micropiles.

3. Repeat computations with the length L_{LONG} reduced in 10 to 15% increments and prepare a graph of ground line deflection versus shaft length as shown schematically in Figure I-1. Smaller length increments may be warranted as the ground line deflection starts to increase.
4. Identify the shaft length, L_O, based on overturning as the length at which the slope of the curve shown in Figure I-1 is approximately zero as identified by change in ground line deflection less than 5% between two consecutive increments of lengths.

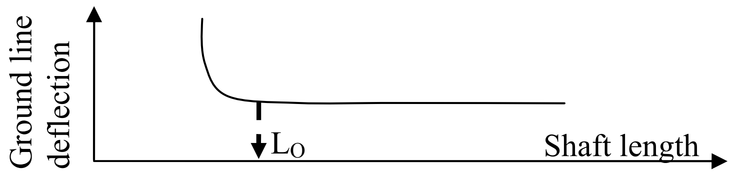

Figure I-1: Use of Ground Line Deflection Curve to Determine L_O.

5. Use the length L_O as an initial value for the evaluation of lateral structural stability based on applicable strength and service limit states with the respective load conditions based on the following considerations:
   a. The lateral structural stability analysis based on applicable strength limit states is used for the structural design of the shaft, e.g., detailing the reinforcement.
   b. The lateral structural serviceability analysis based on applicable service limit states is used for the evaluation of limiting deflections based on serviceability considerations, e.g. the deformation tolerance of the structure being supported by the shaft foundation.
   a. Both strength and service limit state evaluations are based on the use of nonlinear bending stiffness, E_SI_C, where E_S is the composite elastic modulus of the shaft and I_C is the cracked moment of inertia. Refer to Section 5 of AASHTO (2010) for guidance on determination of the nonlinear bending stiffness of concrete members.
6. For the selected shaft diameter, determine the final shaft length based on lateral load analysis, L_L, as the longest of the lengths determined from the evaluations of the overturning strength limit state (for geotechnical stability), structural strength limit state (for structural detailing), and structural service limit state (for serviceability evaluation).
7. As necessary, repeat the above procedure for different shaft diameters to determine the most efficient and cost-effective drilled shaft foundation system. This may involve evaluation of options such as the use of additional shafts that may permit use of reduced shaft lengths, larger diameter shafts for a given length or other means to achieve the required lateral resistance to meet project needs.

It is not possible to identify the limit state that will dictate the governing length without performing all of the analyses described previously since each limit state will be affected by a number of variables including, but not limited to, the configuration of the structure being supported and the subsurface conditions. Specifically, the shaft length based on lateral load analysis, L_L, is dependent on the relative stiffness of the shaft and the geomaterial (soil, rock or intermediate geomaterial) in which the drilled shaft is embedded. Therefore, the choice of the geomaterial resistance parameters is very important. It is prudent to perform parametric studies using a range of values of the geomaterial properties. Such parametric analyses are particularly important for the case of monoshaft foundations where a single shaft supports a substructure unit because such a foundation system is non-redundant.

The final governing design shaft length will be the longer of the lengths based on lateral load and axial load performance requirements and considerations. Guidance for axial load analysis based on LRFD methodology is provided in a separate memorandum (ADOT DS-1, 2010).

II. Minimum Requirements for Analytical Models

Where lateral loads and moments control the drilled shaft design, the final design shall be based on consideration of lateral deformations. Article 10.7.2.4 ("Horizontal Pile Foundation Movement") of AASHTO (2010) provides guidance for the evaluation of horizontal movements induced by lateral loads. Two analytical methods are allowed. These are the "P-y" method[3] and the strain wedge method. Both methods are applicable to a single deep foundation element, e.g., a monoshaft foundation, and a group of deep foundation elements.

The lateral response of a deep foundation group consisting of elements having the same diameter is a function of the ratio of the center-to-center spacing and the diameter of the foundation elements. The P-y method uses a P-multiplier, P_m, to account for group effects. Values of P_m are provided in Table 10.7.2.4-1 of AASHTO (2010). P-multipliers are not applicable to the strain wedge method because that method accounts for group effects through an assessment of the overlap of passive wedges that develop between the deep foundation elements within the group.

Commercial software packages are available for both methods[4]. For a group of shafts, bridge engineers often use a program designed for a single deep foundation element such as COM624P or structural software such as STAAD or RISA because of their familiarity with such software. This practice does not properly model the complex soil-structure interaction that occurs in a deep foundation group and results in unrealistically large deformations and required foundation member sizes. If the shaft heads within a deep foundation group are restrained from rotating, i.e., if they are not pinned, moments will develop at the shaft heads that will induce compressive or tensile loads in the shafts and cause the entire cap to rotate. The sum of the shaft-head moments will be resisted by the sum of the push-pull couples in the shafts within the group and possibly in part by soil passive resistance against the front face of the cap. Any resulting cap rotation will also serve to relieve the moments applied to the shaft heads. In some cases these effects can be significant. Therefore the use of software intended for the analysis of a single shaft or structural frames where these effects cannot be accounted for is not recommended for the analysis of drilled shaft groups. The minimum requirements for the capabilities of a software package to analyze a group of drilled shafts correctly are:

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[3] Conventionally, a lower case p is used in the term "p-y" method. However, AASHTO (2010) uses an upper case P. In this memorandum, the AASHTO convention is followed and "P-y" is used.

[4] Commonly used software packages for the P-y method of analysis of a single deep foundation element include COM624P developed by the Federal Highway Administration (FHWA) and the LPile-series developed by Ensoft Inc. Commonly used software packages for the lateral analysis of a group of deep foundation elements include the GROUP-series developed by Ensoft, Inc. and FB-MultiPier developed by the Bridge Software Institute of the University of Florida. For the strain wedge method, the SWM program distributed by GEOPILE, LLC, can perform analyses for the case of deep foundations in a single or group configuration.

1. Soil-structure interaction is modeled in an integrated manner that accounts for strain-compatible lateral and axial nonlinear soil resistance. The lateral soil resistance is modeled through “p-y” curves while the axial soil resistance is modeled through “t-z” curves for side resistance and “q-w” curves for tip resistance.
2. The nonlinear bending stiffness ($E_S I_C$) of each shaft is modeled.
3. Head conditions that include fixed, pinned or elastically restrained by the group cap are appropriately modeled.
4. The ability to apply vertical loads, horizontal loads and moments simultaneously.
5. The effect of cap resistance is modeled. A rigid cap model is acceptable.
6. In case of a P-y method, the ability to use P-multipliers in accordance with Table 10.7.2.4-1 of AASHTO (2010).

Several tools currently exist within the industry that easily satisfy the above requirements and can be used on routine and complex major projects. For example, GROUP version 8.0, SWM version 6.2, and FB-MultiPier version 4.15 satisfy the above minimum requirements for analysis of a group of drilled shafts. For single shafts, the first four requirements listed above should be satisfied. Commonly available software packages that can perform lateral load analysis of single shafts include, but are not limited to, COM624P, LPile version 6.0, SWM version 6.2, and FB-MultiPier version 4.15.

The Bridge Group and Materials Group of ADOT should be consulted prior to the use of software packages that are not mentioned above. On all projects the bridge and geotechnical specialists of record should interact to ensure that the geotechnical material parameters are in accordance with the computer program that is being considered by the bridge engineer and are appropriate based on the above-noted requirements.

III. Depth to Fixity, D_F

Bridge engineers commonly use the depth to fixity, D_F, because it eliminates the geomaterial considerations from the analysis such that a fixed cantilever-type of structural model can be used. Figure III-1 shows the provisions in AASHTO (2010) for the determination of the depth to fixity. Figure III-2a shows the shaft length, L_L, based on lateral load analysis discussed in Sections I and II of this memorandum. For comparison, Figure III-2b shows the depth to fixity, D_F, according to the provisions in AASHTO (2010), which are based on the work of Davisson and Robinson (1965). While Equations C10.7.3.13.4-1 and C10.7.3.13.4-2 are straightforward and consistent with those in Davisson and Robinson (1965), the parameter that represents the soil resistance, n_h, for sands is not the same as that in the original work [5]. The values in Table C10.4.6.3-2 of AASHTO (2010) shown in Figure III-1 are based on the initial tangent modulus as described in FHWA (1984) while the values of n_h in the original work by Davisson and Robinson (1965) are based on the secant modulus concept developed by Terzaghi (1955). Table III-1 provides the correct values of n_h based on Terzaghi (1955). These values should be used in conjunction with Equation C10.7.3.13.4-2 to be consistent with the original work by Davisson and Robinson (1965).

[5] The guidance for cohesive soils in AASHTO (2010) is appropriate.

10.7.3.13.4—Buckling and Lateral Stability

In evaluating stability, the effective length of the pile shall be equal to the laterally unsupported length, plus an embedded depth to fixity.

The potential for buckling of unsupported pile lengths and the determination of stability under lateral loading should be evaluated by methods that consider soil-structure interaction as specified in Article 10.7.3.12.

C10.7.3.13.4

For preliminary design, the depth to fixity below the ground, in ft, may be taken as:

- For clays:

$$1.4 \left[E_p l_w / E_s\right]^{0.25} \qquad \text{(C10.7.3.13.4-1)}$$

- For sands:

$$1.8 \left[E_p l_w / n_h\right]^{0.2} \qquad \text{(C10.7.3.13.4-2)}$$

where:

E_p = modulus of elasticity of pile (ksi)

l_w = weak axis moment of inertia for pile (ft^4)

E_s = soil modulus for clays = 0.465 S_u (ksi)

S_u = undrained shear strength of clays (ksf)

n_h = rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 (ksi/ft)

This procedure is taken from Davisson and Robinson (1965).

In Eqs. C10.7.3.13.4-1 and C10.7.3.13.4-2, the loading condition has been assumed to be axial load only, and the piles are assumed to be fixed at their ends. Because the equations give depth to fixity from the ground line, the Engineer must determine the boundary conditions at the top of the pile to determine the total unbraced length of the pile. If other loading or pile tip conditions exist, see Davisson and Robinson (1965).

The effect of pile spacing on the soil modulus has been studied by Prakash and Sharma (1990), who found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling resistance. However, at a pile spacing of three times the pile width, the effective soil modulus is reduced to 25 percent of the value applicable to a single pile. For intermediate spacings, modulus values may be estimated by interpolation.

Table C10.4.6.3-2—Rate of Increase of Soil Modulus with Depth n_h (ksi/ft) for Sand

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

Figure III-1: Provisions in AASHTO (2010) Related to Depth to Fixity.

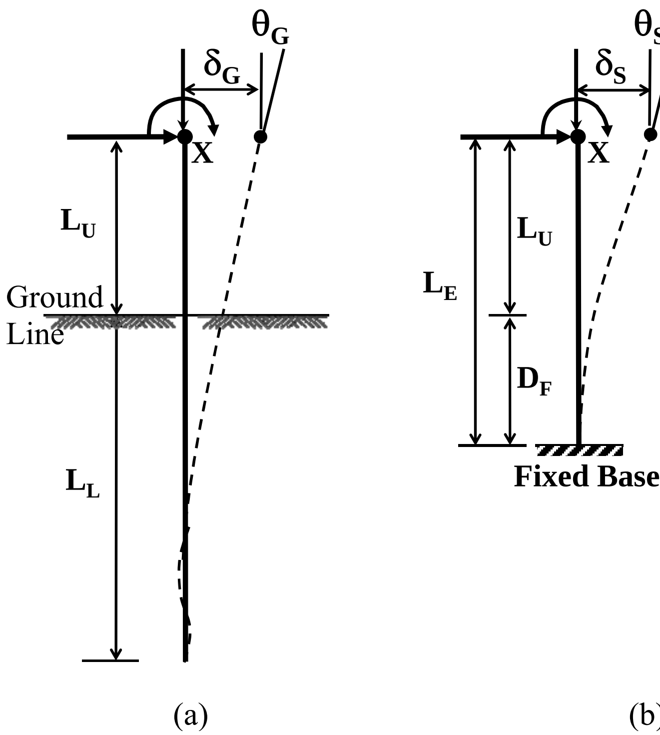

Figure III-2: Definition of Depth to Fixity, D_F. (Note: L_L is determined as discussed in Section I and Point X is the point of application of the moments and loads).

Table III-1
Values of n_h (ksi/ft) for Sand (Terzaghi, 1955)

Consistency	Dry or Moist	Submerged
Loose	0.097	0.056
Medium	0.292	0.194
Dense	0.778	0.472

As shown in Figure III-2b, the equivalent length of the cantilever, L_E, is equal to the unsupported length, L_U, plus the depth to fixity, D_F. The structural member of length L_E in Figure III-2b is presumed to behave in the same manner as the actual soil-structure interaction configuration shown in Figure III-2a. Therefore the equivalent cantilever system in Figure III-2b should be used if the following two conditions are satisfied:

1. Approximately equal deflections at Point X, $\delta_G \approx \delta_S$
2. Approximately equal rotations at Point X, $\theta_G \approx \theta_S$

To achieve the above two conditions, the depth to fixity obtained by using the equations in AASHTO (2010) should be treated as an initial estimate in frame analyses to determine the initial forces, deflections, rotations, and member sizes. These initial forces and member sizes should then be used to determine the drilled shaft lengths based on soil-structure interaction procedures as described in Section I and the deflections and rotations at Point X as shown in Figure III-2a. These deflections and rotations should be compared with the same quantities based on the equivalent cantilever model. If the deflections and rotations are not approximately the same between the two models then the depth to fixity, D_F, should be revised and the procedure should be repeated until both conditions noted above are achieved. Any changes in the depth to fixity, D_F, will also entail revision of the moments and loads. Once both conditions are satisfied, then the depth to fixity based on the converged analyses will yield the correct value.

Note that it may not be possible to concurrently satisfy both conditions exactly. Practically, convergence may be deemed to be achieved if the values being evaluated for both conditions based on the soil-structure interaction model and the equivalent cantilever model are within 5 to 10% of each other.

The equivalent cantilever model based on the converged value of depth to fixity can be used in conventional frame analyses for determining moments and loads at Point X. However, the moment computed for the fixed end of the equivalent cantilever will be considerably larger than that the moment actually occurring along the embedded portion of the shaft, i.e. the length L_L of the constructed shaft. Therefore, to analyze the embedded portion of the shaft correctly it is necessary to use the analytical models described in Section II of this memorandum using the appropriate moments and loads at the ground line. These values can be determined from basic principles of statics once the conditions at Point X have been determined from frame analysis (Davisson and Robinson, 1965).

The traditional depth to fixity approach is typically more appropriate for a monoshaft type of foundation in comparison with a foundation that consists of a drilled shaft group. For a drilled shaft group, the analytical models described in Section II of this memorandum should be used by applying an integrated soil-structure interaction approach.

IV. Considerations for Collapse-Susceptible Soils

Some dry soils in the southwest desert environment are known to undergo rapid volume change (collapse) under moisture ingress and/or stresses due to applied loads. Such collapse-susceptible (metastable) soils can extend from the ground surface to depths of 20- to 30-ft. Since a large portion of the lateral resistance of shafts can be achieved from soils within the upper half of the drilled shaft, the effect of collapse-susceptible soils should be included in the lateral analysis to evaluate the potential for sudden and large vertical and lateral deformations at some time during the service life of the structure. For most applications in the desert southwest where only the top 20-30 feet of the soil profile is expected to contain collapse-susceptible soils that could experience significant moisture ingress, the major effect would be evident by sudden and large lateral deformations.

The following procedure is recommended to account for the presence of potentially collapse-susceptible soils in lateral load analysis:

1. Perform laboratory tests to determine the collapse-potential for soils in accordance with ASTM D5333-03. Use the standard loading sequence noted in ASTM D5333-03 with an applied vertical stress of 4 ksf (kips per square foot) at wetting.
2. Determine the strain at 4 ksf prior to wetting and designate it as, e_p.

3. Determine the strain at 4 ksf after wetting and designate it as e_a.
4. Compute ratio of strains, $CR = e_a/e_p$.
5. For both P-y and SWM approaches, use the CR value as a y-multiplier, y_m, within the depth of the potentially collapse-susceptible soil layer. Use $y_m = 1$, i.e., no modification, in the soil layers below the potentially collapse-susceptible soil layer.

The geotechnical specialist shall perform an adequate number of collapse tests to estimate the thickness of the potentially collapse-susceptible soil layer and to determine a representative value of y_m. As appropriate, subdivide the potentially collapse-susceptible layers with different values of y_m. Provide the value(s) of y_m in the geotechnical report in the section where the recommended soil parameters for lateral load analysis are reported.

The y-multiplier, y_m, is equally applicable to all drilled shafts within a group regardless of the location of a shaft in that group or the direction of loading, i.e., the y-multiplier, y_m, does not account for group effects. Therefore, regardless of the y-multiplier, y_m, the P-multiplier, P_m, from Table 10.7.2.4-1 of AASHTO (2010) to represent group effects shall be applied in accordance with the guidance in Article 10.7.2.4 in AASHTO (2010).

V. Closing Comments

This memorandum contains guidance for the analysis of drilled shafts subjected to lateral loads. In particular, the following steps are anticipated for the proper implementation of the guidance provided in this memorandum:

1. Bridge engineers will use software packages that meet the minimum requirements listed in Section II. In particular, drilled shaft groups will be analyzed by using soil-structure-interaction-based software packages for shaft groups and not by using single shaft software packages as discussed in Section II.
2. Geotechnical specialists will develop and provide the appropriate information on subsurface layers and values for their respective geomaterial properties. Due diligence in the performance of this step is critical because the lateral behavior of deep foundation elements is directly influenced by the relative stiffness of the structural elements and the geomaterials in which they are embedded.
3. Interaction between structural and geotechnical specialists will include an evaluation of the consistency of the soil parameters and the software packages.

Close interaction and communication between geotechnical and bridge specialists will be required to apply the guidance in this memorandum correctly.

VI. References

AASHTO (2010). ***AASHTO LRFD Bridge Design Specifications***. Fifth Edition. American Association of State Highway and Transportation Officials, Washington, D.C. (including latest errata and interims).

ADOT DS-1 (2010). ***Development of Drilled Shaft Axial Resistance Charts for Use by Bridge Engineers Based on Load and Resistance Factor Design (LRFD) Methodology.*** Memorandum from N. H. Wetz and J. D. Wilson to J. Lawson, Dated December 1, 2010, Arizona Department of Transportation. Phoenix, AZ. (http://www.azdot.gov/Highways/Materials/Geotech_Design/Policy.asp)

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FHWA (2006). ***Soils and Foundations – Volumes I and II***. Authors: Samtani, N. C. and Nowatzki, E. A., Publications No. FHWA NHI-06-088 and FHWA NHI-06-089, Federal Highway Administration, Washington, D.C.

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