AGENDA ITEM 39 - ATTACHMENT

Modify Table 3.4.1-2 in Article 3.4.1 regarding the downdrag load factor as follows:

Table 3.4.1-2 – Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DD: Downdrag	Piles, α-Tomlinson Method	1.4	~~--~~ 0.25
	Piles, λ-Method	1.05	~~--~~ 0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	~~--~~ 0.35

Replace Article 3.11.8 and commentary with the following:

3.11.8 Downdrag

Possible development of downdrag on piles or shafts shall be evaluated where:

- Sites are underlain by compressible material such as clays, silts or organic soils,
- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills,
- The groundwater is substantially lowered, or
- Liquefaction of loose sandy soil can occur

When the potential exists for downdrag to act on a pile or shaft due to downward movement of the soil relative to the pile or shaft, and the potential for downdrag is not eliminated by preloading the soil to reduce downward movements or other mitigating measure, the pile or shaft shall be designed to resist the induced downdrag.

Consideration shall be given to eliminating the potential for downdrag loads through the use of embankment surcharge loads, ground improvement techniques, and/or vertical drainage and settlement monitoring measurements.

For Extreme Event I limit state, downdrag induced by liquefaction settlement shall be applied to the pile or shaft in combination with the other loads included within that load group. Liquefaction-induced downdrag shall not be combined with downdrag induced by consolidation settlements.

For downdrag load applied to pile or shaft groups, group effects shall be evaluated.

C3.11.8

Downdrag, also known as negative skin ~~resistance~~ friction, can be caused by soil settlement due to loads applied after the piles were driven, such as an approach embankment as shown in Figure C1. Consolidation can also occur due to recent lowering of the ground water level as shown in Figure C2.

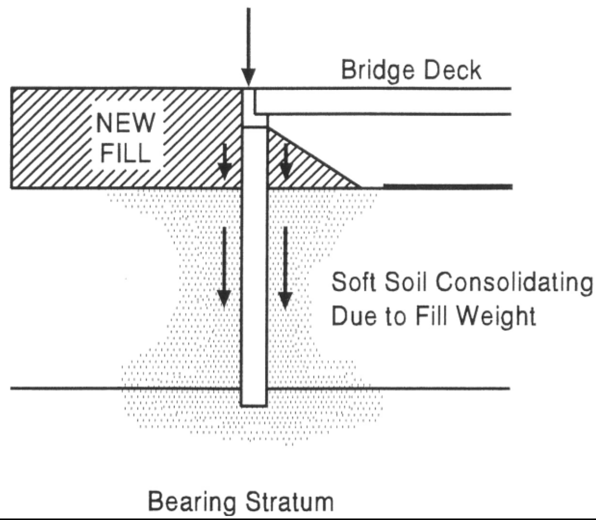

Figure C3.11.8-1 – Common Downdrag Situation Due to Fill Weight (Hannigan, et al. 2005)

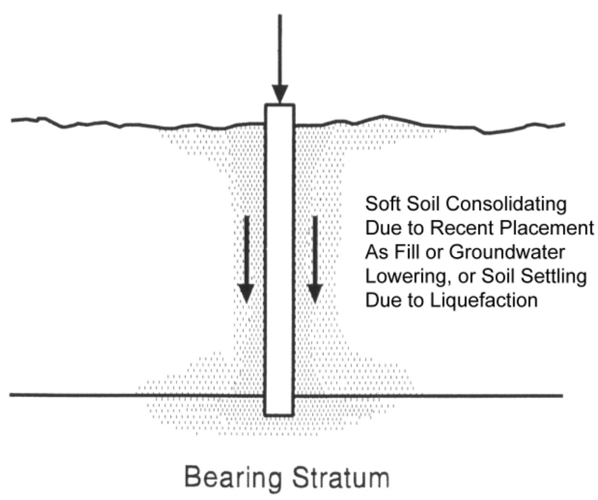

Figure C3.11.8-2 – Common Downdrag Situation Due to Causes Other Than Recent Fill Placement

~~Regarding the load factors for downdrag in Table 3.4.1-2, only maximum load factors are presented. If downdrag is acting as a restoring~~

Proposed Specification	Commentary
	~~force (e.g., the pile or shaft is acting to resist uplift forces), the downdrag should be treated as an uplift resistance, and an appropriate uplift resistance factor should be selected from Article 10.5.5.2.~~ Regarding the load factors for downdrag in Table 3.4.1-2, use the maximum load factor when investigating maximum downward pile loads. ~~and t~~The minimum load factor shall only be utilized when investigating possible uplift loads. For some downdrag estimation methods, the magnitude of the load factor is dependent on the magnitude of the downdrag load relative to the dead load. The downdrag load factors were developed considering that downdrag loads equal to or greater than the magnitude of the dead load become somewhat impractical for design. See Allen (2005) for additional background and guidance on the effect of downdrag load magnitude. Methods for eliminating static downdrag potential include preloading. The procedure for designing a preload is presented in Cheney and Chassie (2000). Post-liquefaction settlement can also cause downdrag. Methods for mitigating liquefaction-induced downdrag are presented in Kavazanjian, et al. (1997). The application of downdrag to pile or shaft groups can be complex. If the pile or shaft cap is near or below the fill material causing consolidation settlement of the underlying soft soil, the cap will prevent transfer of stresses adequate to produce settlement of the soil inside the pile or shaft group. The downdrag applied in this case is the frictional force around the exterior of the pile or shaft group and along the sides of the pile or shaft cap (if any). If the cap is located well up in the fill causing consolidation stresses or if the piles or shafts are used as individual columns to support the structure above ground, the downdrag on each individual pile or shaft will control the magnitude of the load. If group effects are likely, the downdrag calculated using the group perimeter shear force should be determined in addition to the sum of the downdrag forces for each individual pile or shaft. The greater of the two calculations should be used for design. The skin friction used to estimate downdrag due to liquefaction settlement should be conservatively assumed to be equal to the residual soil strength in the liquefiable zone, and nonliquefied skin friction in nonliquefiable layers above the zone of liquefaction.
If transient loads act to reduce the magnitude of downdrag loads and this reduction is considered in the design of the pile or shaft, the reduction shall not exceed that portion of transient load equal to the downdrag force effect.	Transient loads can act to reduce the downdrag because they cause a downward movement of the pile resulting in a temporary reduction or elimination of the downdrag load. It is conservative to include the transient loads together with downdrag.

Proposed Specification	Commentary
Force effects due to downdrag on piles or drilled shafts should be determined as follows: Step 1 – Establish soil profile and soil properties for computing settlement using the procedures in Article 10.4.	The step-by-step procedure for determining downdrag is presented in detail in Hannigan, et al. (2005).
Step 2 – Perform settlement computations for the soil layers along the length of the pile or shaft using the procedures in Article 10.6.2.4.2.	The stress increases in each soil layer due to embankment load can be estimated using the procedures in Hannigan et al. (2005) or Cheney and Chassie (2000).
Step 3 – Determine the length of pile or shaft that will be subject to downdrag. If the settlement in the soil layer is 0.4 in. or greater relative to the pile or shaft, downdrag can be assumed to fully develop.	If the settlement is due to liquefaction, the Tokimatsu and Seed (1987) or the Ishihara and Yoshimine (1992) procedures can be used to estimate settlement.
Step 4 – Determine the magnitude of the downdrag, DD, by computing the negative skin resistance using any of the static analysis procedures in Article 10.7.3.7.5 for piles in all soils and Article 10.8.3.3.1 for shafts if the zone subject to downdrag is characterized as a cohesive soil. If the downdrag zone is characterized as a cohesionless soil, the procedures provided in Article 10.8.3.3.2 should be used to estimate the downdrag for shafts. Sum the negative skin resistance for all layers contributing to downdrag from the lowest layer to the bottom of the pile cap or ground surface. The neutral plane method may also be used to determine downdrag.	The methods used to estimate downdrag are the same as those used to estimate skin friction, as described in Articles 10.7 and 10.8. The distinction between the two is that downdrag acts downward on the sides of the piles or shafts and loads the foundation, whereas skin friction acts upward on the sides of piles or shafts and, thus, supports the foundation loads. Downdrag can be estimated for piles using the α or λ methods for cohesive soils. An alternative approach would be to use the β method where the long-term conditions after consolidation should be considered. Cohesionless soil layers overlying the consolidating layers will also contribute to downdrag, and the negative skin resistance in these layers should be estimated using an effective stress method. Downdrag loads for shafts may be estimated using the α-method for cohesive soils and the β-method for granular soils, as specified in Article 10.8, for calculating negative shaft resistance. As with positive shaft resistance, the top 5.0 ft. and a bottom length taken as one shaft diameter do not contribute to downdrag loads. When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. The neutral plane method is described and discussed in NCHRP 393 (Briaud and Tucker, 1993).

Add the following references to Section 3 to accommodate the changes in Article 3.11.8:

REFERENCES

Allen, T. M., 2005, *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, Publication No. FHWA-NHI-05-052, Federal Highway Administration, Washington, DC, 41 pp.

Briaud, J. and Tucker, L. 1993. NCHRP 393/Project 24-05, Downdrag on Bitumen-Coated Piles.

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Hannigan, P.J., G.G.Goble, G. Thendean, G.E. Likins and F. Rausche 2005. "Design and Construction of Driven Pile Foundations" - Vol. I and II, Federal Highway Administration Report No. FHWA-HI-05, Federal Highway Administration, Washington, D.C.

Ishihara, K., and Yoshimine, M. (1992). *Evaluation of settlements in sand deposits following liquefaction during earthquakes.* Soils and Foundations, JSSMFE, Vol. 32, No. 1, March, pp. 173-188.

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Tokimatsu, K. and Bolton Seed, B. 1987. Evaluation of Settlements in Sands due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, 113, 8, 861-878.