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Introduction

The installation of new foundations and the modification of existing foundations for telecommunication structures represent significant capital investment costs. In many instances, limited space at the site or relatively poor soil conditions require the use of specialized foundation systems or layouts that increase these construction costs. The resulting designs may require foundations with large footprints, deep installation depths, and/or the need for soil anchors when space is limited. Based on these factors, the telecom industry is impacted when excessive foundation costs limit investment and expansion by wireless carriers.

When designing or analyzing foundations the primary structural considerations are the following: 1) stability of the foundation against uplift, sliding and overturning, 2) bearing pressures on the soil, and 3) limit state checks of the reinforced concrete members or embedded steel sections. TIA-222-H provides design criteria for these limit states based solely on LRFD design methodologies.

However, ASCE 7-16 and IBC 2018 allow for the use of LRFD <u>or</u> ASD methodologies when analyzing foundations. When evaluating the stability and soil bearing pressures of mat or pad type foundations in can be beneficial to consider using ASD load combinations for determining the eccentricity of the loading. In this manner the stability and soil bearing capacity of the foundation can often be acceptable using ASD when a LRFD approach would deem the foundation inadequate.

Another common assumption for foundations analysis is rigid body behavior. While this assumption allows for relatively simple calculations, it may not correlate well with the actual behavior and may in fact penalize the foundation design. An analysis that considers the flexible behavior of the foundation and stiffness of the soil can better approximate the true behavior and yield a more efficient design. This is especially true in the case of drilled pier foundations with large lateral and overturning forces.

Consequently, current industry practice for foundation analysis may result in conservative or inefficient designs. Therefore, alternate methods of analysis are needed to find more cost-effective solutions. Specifically, this paper advocates the use of ASD load combinations when reviewing stability of foundations and the consideration of performance-based design when analyzing drilled pier foundations.

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Stability Analysis-Mat and Pad & Pier Foundations

The stability of mat and pad and pier foundations rely upon the weight of the structure, foundation and any soil to resist the overturning forced due to lateral loads on the structure. Focusing on dead loads and wind loads and ignoring other load effects, the critical LRFD load combinations from section 2.32 of TIA-222-H that must be considered simplify to the following:

$$1.2D+1.0W$$

$$0.9D+1.0W$$

where,

D=Dead Load

W=Wind load

Using the above equations, a foundation is considered to be stable if the eccentricity of loading (gross overturning moment divided by gross axial load) calculated from the above equations is less than 45% of the foundation width in the direction of loading.

Prior to Revision G of TIA-222, ASD equations were utilized in TIA. The equation of note in TIA-222-F was:

$$1.0D+1.0W$$

When determining the stability of a footing using this ASD equation an additional factor of safety of 1.5 was required when comparing the overturning moment to the overturning resistance of the foundation. The current TIA standard no longer explicitly requires this factor of safety. It is important to realize that the current wind speeds are based on ultimate wind speeds which are approximately 1.6 times greater than the nominal wind speeds that were used in previous versions of the standard. For comparison purposes, an equivalent factor of safety that can be obtained from the current TIA-222-H standard by considering the 1.6 wind load factor and 0.9 dead load factor is:

$$F.O.S.= 1.6/0.9 = 1.78$$

Section 2.3.1 of ASCE 7-16 and Section 1605.2 of IBC have similar LRFD equations that simplify to be equivalent to the equations in TIA-222-H when only the applicable load cases are considered. At the same time, ASCE 7-16 section 2.4.1 and IBC 2018 section 1605.3 have ASD equation that simplify to the following:

$$1.0D+0.6W$$

$$0.6D+0.6W$$

From a stability standpoint the second equation is generally the most critical. It should be noted that the 0.6 factor on the wind load is to bring the forces in line with nominal rather than ultimate wind speeds, as necessary for use with ASD. The 0.6 factor on the dead load is an effort to insure stability. Converting to a factor of safety:

F.O.S. = 1/0.6 = 1.67

Also, note that in previous version of IBC the load factor on the dead load had been 0.9. It was reduced to 0.6 so that the 1.5 factor of safety could not be accidentally omitted.

Therefore, the resulting factor of safety using either the ASD or LRFD equations of ASCE and IBC are still greater than the 1.5 factor that was required by earlier versions of TIA. No additional factor of safety is required for use with the current ASD equations of IBC and ASCE 7. It should also be noted that ASCE 7 and the IBC 2018 do not limit the use of ASD or LRFD load combinations for stability and soil checks. Section 1605.1.1 allows the use of either ASD or LRFD load combinations when assessing stability. In addition, Section 1802.1 of the IBC 2018 states the following, "*Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3.* This confirms that the use of ASD methodologies is acceptable for foundation analysis and should be considered as an acceptable for use in the design and analysis of telecommunication foundations.

Example 1

Using the ASD equations can result in better results for a foundation. As an example, consider a foundation that is 20'x20', with a total unfactored dead load of the structure, foundation and soil of 200 kips. The maximum acceptable eccentricity for a 20 ft foundation is 9 ft. The equation for eccentricity, e=M/P can be rearranged to solve for the maximum applied moment, Mmax=P*e.

LRFD: 0.9D+1.0W

Mmax=0.9*200k*9ft=1620 k-ft

ASD: 0.6D+0.6W

0.6*Mmax=(0.6*200*9ft)=1080 k-ft

For comparison to LRFD level loads:

Mmax=1080 k-ft / (0.6) = 1800 k-ft

Thus, in this case it can be shown that the ASD methodology provides for an increase of 11% in the maximum overturning moment that can be applied to the foundation before it becomes unstable.

The ASD equations may also be beneficial in cases where soil bearing controls instead of stability. From the previous example, assuming the factored moment of 1620 k-ft is applied, eccentricity and bearing pressures for LRFD and ASD can be determined as follows:

LRFD:

e=M/P=1620/(0.9*200)=9 ft

e'=L/2-e=20/2-9=1 ft

q=2*P/(3*B*e')=2*(0.9*200)/(3*20*1)= 6 ksf

ASD:

e=M/P=(0.6*1620)/(0.6*200)=8.1 ft

e'=L/2-e=20/2-8.1=1.9 ft

q=2*P/(3*B*e')=2*(0.6*200)/(3*20*1.9)= 2.11 ksf

Assuming the common phi factor of 0.75 for LRFD and safety factor of 2 for ASD, the required ultimate bearing pressure can be determined:

LRFD: Required ultimate bearing=6/(0.75)=8 ksf

ASD: Required ultimate bearing pressure=2.11*2=4.22 ksf.

This shows that ASD methodologies can also provide significant additional capacities when ultimate soil bearing pressures control the foundation capacity.

In practice, the following steps are proposed:

1. Determine base reactions for the structure based on the ultimate wind speeds per the TIA-222-H standard.
2. Utilize the base reactions and ASD load combinations to determine eccentricity and soil bearing pressures. This would result in a free body diagram (FBD) based on ASD loading.
3. Check stability and allowable soil bearing pressures.
4. Factor the forces from the ASD FBD by the required load factors to complete the design of reinforced concrete or any embedded steel cross sections per LRFD methodologies of TIA-222-H.

Performance Based Design – Deep foundations

Section 1810.2.4 of IBC 2018 indicates that forces and deflections in deep foundations shall be established considering the nonlinear interaction of the shaft and soil. However, there is as exception that it is acceptable for foundations to be considered as rigid when the ratio of depth of embedment to least foundation dimension is less than six (L/d<6). It is important to note although the rigid simplification is allowed for foundation with L/d< 6, it is still permitted to complete an analysis that considers the nonlinear interaction.

Common industry practice is to assume rigid behavior for any foundation where L/d<6. The reason for this is that the analysis can be simplified into equations that can easily be used in spreadsheets, eliminating the need for more complicated analysis in other analysis software. The method developed by Broms is one of the most widely accepted methodologies used within the industry for rigid analysis of deep foundations. The use of Broms is considered by many to be a simplified and preliminary design methodology. Outside of the telecommunications industry, the use of Broms method for final design is typically not recommended.

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Foundations should be analyzed using a soil structure interaction approach (i.e. p-y method) of a program that considers the lateral stiffness of the soil, such as a nonlinear computer-based program. It is important to note that the output of a such a program will typically provide internal forces in the foundation as well as lateral deflection of the foundation. However, it will not provide output capacities for the soil that clearly define that the soil is passing or failing. Rather, the engineer must consider the tower's tolerance for foundation deflections. This load-deflection relationship represents the critical consideration in a performance-based design methodology.

The primary guidance the that TIA provides regarding analyses that consider lateral stiffness of the soil can be found in section 9.7. If soil stiffness is modeled, it requires that the factored reactions are divided by the phi factor. In the case of typical drilled piers with lateral loads, this equates to a phi of 0.75. The result is that the factored reactions need to be increased by 1.33 for the p-y method analysis. As previously stated regarding ultimate and nominal wind speeds, this is equivalent to a total factor of safety on the nominal loads of:

Factor of safety = (1/0.75)*(1/0.6)=2.22

Compared to the requirements of other industries, the TIA requirements are conservative. For example, the 2016 Interim Revisions to the AASTHO LRFD Bridge Design Specifications, Seventh Edition 2014 do not require the LRFD reactions to be increased in a similar manner. The commentary of section 10.7.3.12 state: "When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the p-y curves are <u>not</u> factored since they already represent the ultimate condition". The use of p-y curves results in a more accurate analysis of the soil behavior, so additional phi factors do not need to be applied. Furthermore, the resistance factor (phi) for Horizontal Geotechnical Resistance of a shaft is 1.0 per Table 10.5.5.2.4-1.

For comparison purposes, the Strength III load combination of AASHTO Table 3.4.1-1 would be the equivalent equations to those of TIA-222-H. Note that the ultimate wind speeds used in AASHTO for Strength combinations are similar to the ultimate wind speeds of ASCE 7-10. Simplifying those equations to eliminate load effects that do not apply results in the following equations:

0.9*DC + 1.0 WS

1.25*DC + 1.0 WS

Where

DC=dead load of structural components and non-structural attachments

WS=Wind load on structure.

Similarly, for comparison of serviceability the equivalent AASHTO equations would be Service I calculated based on a wind speed of 70mph. The equivalent equation is:

1.0*DC + 1.0 WS.

As a result, AASTHO has an effective Factor of Safety for the strength of the soil that relies solely on the 1.6 load factor implicitly built into the ultimate wind speeds. At the same time, the AASTHO requirements establish that it is critical that horizontal movement of the foundation be considered. Per AASHTO section 10.5.2.2, "Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement." A specific dimension for horizontal deflection is not listed in AASTHO, rather it is based on the structure's ability to accommodate the anticipated movement at the given site.

Per section 1810.3.3.2 of 2018 IBC, the allowable lateral pile resistance is defined based on maximum allowable deflection. The deflection is determined based on field tests or an analysis that confirms that the foundation deflections do not cause harmful distortion or insatiability of the structure. Typically, if only Broms method is utilized for the foundation analysis then the foundation deflections are not calculated. In these types of Broms only analysis cases, a minimum factor of safety of 2 is required. In order to implement an equivalent factor of safety in a LRFD methodology a phi factor of 0.75 should be applied to lateral soil resistance parameters used in a Broms analysis

Currently the TIA only has deflection limits for the foundation under the serviceability load case as specified in Section 9.4. If the structure is supported by a single caisson foundation, or other site-specific critical foundation, the maximum lateral deflection shall be 0.75" maximum under the serviceability limit state load combination. This requirement can easily be checked in most software utilizing the p-y method of analysis.

However, since it is being recommended that the effective penalty of 1/0.75 factor per TIA Section 9.7 be removed for performance-based design, it is proposed to perform additional deflection checks of the structure and foundation to ensure suitable performance of the structure. The additional checks are based on the requirements of TIA Section 2.8.2. Specifically, it should be confirmed that the additional effects of the foundation deflection and rotation do not result in the 4-degree rotation or 3% deflection of the tower being exceeded under serviceability loading conditions.

The process to check the structure requires the use of a p-y foundation analysis program to determine the pile head deflection and rotation for the service load combination. After obtaining these values, the model of the above grade structure should be created in 3D analysis software that allows for nodes of the model to be moved and rotated to account for the pile deflections. After these adjustments are made, the model can be run for serviceability loads to confirm that the limits of 2.8.2 have still been met.

Strength limit states for the structure may also require additional checks to account for foundation deflection and rotation. The p-y software analysis should be completed considering the factored loads without any adjustment due to the phi factor. The effect of the pile head rotation can then be applied to the 3D model of the structure for the ultimate loading. Important checks for this model include confirming structure forces are still acceptable and that the resulting base reactions do not significantly increase due to rotations. If the increase in base reactions are determined to be too large for this step, multiple iterations may be required to show that the increase is negligible form the previous iteration, and to confirm convergence. However, it may be possible to ignore secondary effects on the structure

for relatively small foundation deflections under the ultimate design loads. The goal is to determine a consensus for these limits and define criteria in the TIA standard.

An important consideration for this performance-based approach is consideration of the soil-structure input parameters. A critical assumption is that the input parameters for the p-y analysis are based on deflection of the soil and that a considerable margin of safety is incorporated into the values to prevent a strength failure of the soil. As a result, the geotechnical parameters provided for use on the foundation analysis should be determined by a geotechnical engineer based on site specific criteria and with knowledge of use in the performance-based design. If the existing information is inadequate, then additional soil investigations may be required. The use of information from old reports based on differing analysis assumptions and methodologies may result in inaccurate results.

In practice, the following steps are proposed:

Step A:

1. Determine base reactions for the structure based on the ultimate wind speeds per the TIA-222-H standard.
2. Analyze foundation using p-y method foundation software to confirm model convergence and determine foundation rotation and deflection. The parameters should be provided or confirmed by an experienced Geotechnical engineer.
3. Create 3D model of the structure to incorporate rotations.
4. Run model at ultimate wind speed and check strength limit state of tower, plates and anchors.
5. Confirm that increase in base reactions are acceptable and represent convergence.
6. Iterate as required.

Step B:

1. Determine base reactions for the structure based on the serviceability wind speeds per the TIA-222-H standard.
2. Analyze foundation using p-y method foundation software to determine foundation rotation and deflection. Confirm less than 0.75”.
3. Create 3D model of the structure to incorporate rotations.
4. Run model at serviceability wind speed and check deflection limits states of TIA Section 2.8.2.
5. For any microwave antennas, confirm that the rotation (twist and sway) of the structure meets the requirements of Annex D or other owner criteria.

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Conclusions

New foundations or modifications to existing foundations continue to be a substantial financial commitment for tower owners. Factors such as soil conditions and site constraints contribute to the costs. Based on these significant costs and site limitations, the goal is to introduce analysis procedures that can more accurately model foundation behavior in order to reduce the costs associated with new foundations and foundations for telecommunication structures.

Recommendation for consideration include:

1. Allowing ASD load combinations (unfactored) to be used for determination of foundation stability and soil bearing pressures.
2. Utilizing performance-based design for deep foundation rather than Broms method in order to more accurately model the actual behavior of the foundation.
3. Eliminating the 1/phi increase of factored reactions for performance-based analysis to maintain a more realistic representation of foundation and structural deflections.

These analysis methodologies have been successfully used for foundation analysis and design in other industries. The IBC code and ASCE and AASTHO standards allow for and support these proposed changes. Allowing for use of these methods in TIA standard would result in cost savings for foundations on many sites, especially if foundation modifications can be avoided to existing foundations.