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STABILITY AND PERFORMANCE CRITERIA FOR FOUNDATIONS

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CHAPTER I
INTRODUCTION

The installation of new foundations and the modification of existing foundations for telecommunication structures represent significant capital investment costs. This capital invested in costly foundation installations and modifications limits the use of that capital for other telecommunications infrastructure. For the industry to support the demand for telecommunications infrastructure as effectively as possible, all analysis methods that can minimize cost should be considered. This white paper is intended to encourage the effective foundation design in accordance with the codes and standards that apply while allowing options to be considered that can avoid un-necessary costs due to simplified design methods.

In many instances, limited space at the site or relatively poor soil conditions can require the use of specialized foundation systems or layouts that increase construction costs. The resulting designs may require foundations with large footprints, deep installation depths, or the need for soil anchors when space is limited. Based on these factors, the telecommunications industry is impacted when excessive foundation costs limit investment, network redundancy, and technology upgrades by wireless carriers.

Rigid body behavior is a typical assumption for foundations analysis in the Telecommunications Industry. While this assumption allows for relatively simple calculations, it may not correlate well with the actual behavior and may in fact penalize the calculated foundation capacity. An analysis that considers the flexible behavior of the foundation and stiffness of the soil can better assess the true behavior to allow optimized results. This is especially true in the case of drilled pier foundations with large lateral and overturning forces.

Current industry practice also typically analyzes the tower using a model that assumes the supporting foundation is fully fixed or pinned. The support reactions resulting from this analysis are then applied to the foundation in a separate set of calculations that do not consider the impact of foundation movement on the structure. Due to this separation in the analysis process, some assumptions must be made for the foundation analysis that may not allow the full capacity of the foundation to be realized. The method proposed in this paper requires a comprehensive review of the combined behavior of the foundation and structure that allows the foundation capacity to be optimized.

Consequently, current industry practices for foundation analysis may result in sub-optimal designs. Therefore, alternate methods of analysis are needed to find more cost-effective solutions. Specifically, this paper advocates for the consideration of performance-based analysis for foundations. It is likely that this will require some additional engineering effort; however, the engineering costs will often prove to be considerably less than costs associated with the foundation installation or upgrade that would otherwise be required when using other analysis methods.

CHAPTER II
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 $L/d<6$. This results in an analysis that can be simplified into equations that are easily 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 in many industries to be a simplified and preliminary design methodology. Outside of the telecommunications industry, the use of Broms method for final design is typically not utilized.

Analysis of foundations using a soil structure interaction approach (i.e. p-y method) of a program that utilizes the lateral stiffness of the soil, such as a nonlinear computer-based program, should be considered by the engineer. 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 account for the tower's foundation deflection tolerance. This load-deflection relationship represents the critical consideration in a performance-based design methodology.

The primary guidance that ANSI/TIA-222-H 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 resistance factor, ϕ. 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.

Compared to the requirements of other industries, the ANSI/TIA-222-H requirements are conservative. For example, the 2016 Interim Revisions to the AASTHO LRFD Bridge Design Specifications, 9th Edition 2020 do not require the LRFD reactions to be increased in a similar manner. The commentary of section 10.7.3.12 states: "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 not 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 the AASTHO equations to eliminate load effects that do not apply to telecommunication structures 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.

It should be noted that the resulting forces from these equations are analogous to the ANSI/TIA loading requirements without the need to increase the structure's base reactions by 1.33 for p-y method analysis (i.e. 1/0.75). At the same time, the AASTHO requirements establish that it is critical that the 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 2021 IBC, the allowable lateral pile resistance is defined based on the 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 instability of the structure.

Currently, the ANSI/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 in. 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 increase of 1.33 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 from 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. Future studies may be required in order to determine a consensus for the limits where secondary effects may be neglected and define criteria in the ANSI/TIA standard.

An important consideration for this performance-based approach is the consideration of the soil-structure input parameters. A critical assumption is that the input parameters for the p-y analysis are based on the 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 in 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 ANSI/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 states of tower, connection plates and anchors.
5. Confirm that any 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 ANSI/TIA-222-H standard.
2. Analyze foundation using p-y method foundation software to determine foundation rotation and deflection. Confirm less than 0.75 in.
3. Create 3D model of the structure to incorporate rotations.
4. Run model at serviceability wind speed and check deflection limits states of ANSI/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.

CHAPTER III
A WORKING EXAMPLE

The following example intends to compare two different analysis methodologies and show the details for the application of a performance-based approach and the potential benefits. The example examines a common foundation type and size with assumed soil parameters where the foundation is considered to be failing for over-turning using a Broms-based rigid-body approach. The selected foundation embedment (L/D<6) does not require flexible analysis per section 9.4 of the current ANSI/TIA-222-REV H, therefore the standard Broms or performance based analysis methodologies can be applied.

Example Site Parameters:

- Foundation size - 5 ft diameter caisson with an 18 ft embedment (L/D = 3.6).
  - 12 #11 bars for vertical reinforcing with #5 ties at 12-inch spacing.
  - The concrete compression strength is 3 ksi.
- Design soil – the internal angle of friction of 30 degrees, soil unit weight of 105 PCF.
- Applied ultimate base reactions – Moment of 2002.6 kip-ft, Shear of 24.4 kips, Axial of 15.3 kips.
- Applied service base reactions – Moment of 300.4 kip-ft, Shear of 4.1 kips, Axial of 12.8 kips.

Performance-based procedure for worked example:

<u>Ultimate Load Case</u>
Step A1. See above and Table 9-1 for the original base reactions for this example.
Step A2. Using the given information, a p-y method analysis of the foundation is completed (LPILE software used in this example). The results of an LPILE analysis and associated foundation rotations are provided in Table 9-2.
Step A3. Based on the rotations from Step A2, the tower model is rotated accordingly. See Figure 9-3 for a sketch of the tower rotated about its base in a RISA-3D model.

Step A4. The tower model is analyzed with the rotations from the first iteration of LPILE. New reactions are noted as iteration 2 in Table 9-1. Note that in this example, the capacity utilization of the tower increases from 92.79% to 93.76%.
Step A5. The base moment has increased from 2002.58 k-ft to 2023.63 k-ft. This represents an approximate increase of 1% in the base reactions due to foundation rotations.
Step A6. Steps A2 through A5 are repeated to ensure convergence. Note that in this example, the third iteration resulted in negligible increases in pile deflection and base reactions; therefore, convergence is confirmed. See iteration 3 of Table 9-1 and 9-2 for the final relative change between iterations.

<u>Service Load Case</u>
Step B1. See above and Table 9-4 for the original base reactions for this example.
Step B2. Using the given information, a p-y method analysis of the foundation is completed. The results of an LPILE analysis and associated foundation rotations are provided in Table 9-5. Note that pile head deflection is much less than 0.75 in.
Step B3. Based on the rotations from Step B2, the tower model is rotated accordingly. See Figure 9-4 for a sketch of the tower rotated about its base in a RISA-3D model. Note that rotation of the foundation results in lateral movement at the top node of the tower of 0.163 ft. for the unloaded model, which is approximately 2 in.
Step B4. The tower model is analyzed with the rotations from the first iteration of LPILE. RISA-3D deflections and rotations are noted in Table 9-6. In the case of the first iteration, the rotation at the top of tower that should be considered is 0.6461deg, which is much less than the 4 degree limit. Similarly, the total horizontal movement that should be considered is 7.4163 in. which is also much less that the limit of 3% of the tower height, which is 36 in. for this 100 ft. tower.
Step B5. Steps B2 through B4 are repeated to ensure convergence. Note that in this example, the third iteration resulted in negligible increases in pile deflection and base reactions; therefore, convergence is confirmed. See iteration 3 of Table 9-5 and 9-6 for the final relative change between iterations.

Table 9-1: Tower Demand/Capacity (Ultimate)

TOWER DEMAND/CAPACITY (ULTIMATE)					
Iteration	**Mu (kip*ft)**	**Pu (kip)**	**Vu (kip)**	**Tower Capacity (%)**	**Relative Change (%)**
Original	2002.5833	15.3439	24.4095	92.79%	-
1	2002.5833	15.3439	24.4095	92.79%	0.000
2	2023.6282	15.3439	24.4095	93.76%	1.045
3	2024.273	15.3439	24.4095	93.79%	0.032

Table 9-2: Foundation Deflection/Rotation (Ultimate)

FOUNDATION DEFLECTION/ROTATIONS (ULTIMATE)					
Iteration	**Pile-Head Deflection (in.)**	**Relative Change (%)**	**Pile-Head Rotation (rad)**	**Pile-Head Rotation (deg)**	**Relative Change (%)**
Original	0.0000	-	-	0.0000	-
1	4.9408	100.000	0.03932	2.2530	100.000
2	5.0986	3.193	0.04063	2.3279	3.325
3	5.1035	0.096	0.04067	2.3303	0.100

Table 9-3: Tower Deflection/Rotation (Ultimate)

TOWER DEFLECTION/ROTATIONS (ULTIMATE)					
Iteration	Tower Deflection (in)	Relative Change (%)	Tower Rotation (rad)	Tower Rotation (deg)	Relative Change (%)
Original	52.4091	-	0.082084	4.7031	-
1	52.6438	0.446	0.082343	4.7179	0.315
2	52.6488	0.009	0.082348	4.7182	0.006
3	52.6489	0.0002	0.082349	4.7183	0.001

Table 9-4: Tower Demand/Capacity (Service)

TOWER DEMAND/CAPACITY (SERVICE)					
Iteration	Mu (kip*ft)	Pu (kip)	Vu (kip)	Tower Capacity (%)	Relative Change (%)
Original	300.4257	12.7866	4.0931	14.33%	-
1	301.3089	12.7866	4.0931	14.37%	0.278
2	301.3108	12.7866	4.0931	14.37%	0.000
3	301.3108	12.7866	4.0931	14.37%	0.000

Table 9-5: Foundation Deflection/Rotation (Service)

FOUNDATION DEFLECTION/ROTATIONS (SERVICE)					
Iteration	Pile-Head Deflection (in.)	Relative Change (%)	Pile-Head Rotation (rad)	Pile-Head Rotation (deg)	Relative Change (%)
Original	0.0000	-	-	0.0000	-
1	0.2213	100.000	0.00162	0.0930	100.000
2	0.2218	0.249	0.00163	0.0932	0.256
3	0.2218	0.001	0.00163	0.0932	0.001

Table 9-6: Tower Deflection/Rotation (Service)

TOWER DEFLECTION/ROTATIONS (SERVICE)					
Iteration	Tower Deflection (in)	Relative Change (%)	Tower Rotation (rad)	Tower Rotation (deg)	Relative Change (%)
Original	7.4016	-	0.01126	0.6452	-
1	7.4163	0.198	0.011276	0.6461	0.142
2	7.4164	0.001	0.011277	0.6461	0.009
3	7.4164	0.000	0.011277	0.6461	0.000

Figure 9-1: Modified Brom's Load Diagram

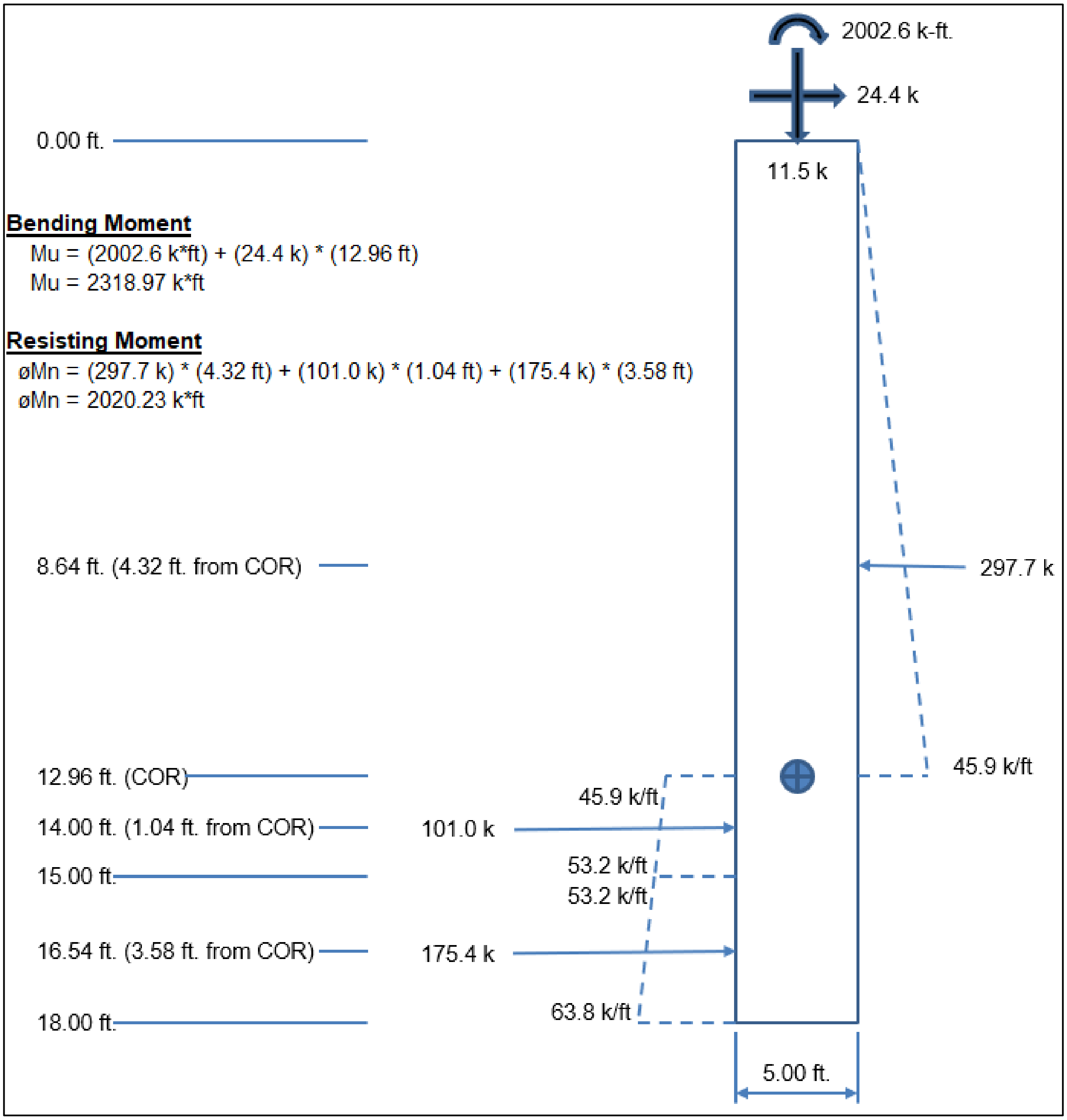


Figure 9-2: LPILE Lateral Pile Deflection (Ultimate & Service)

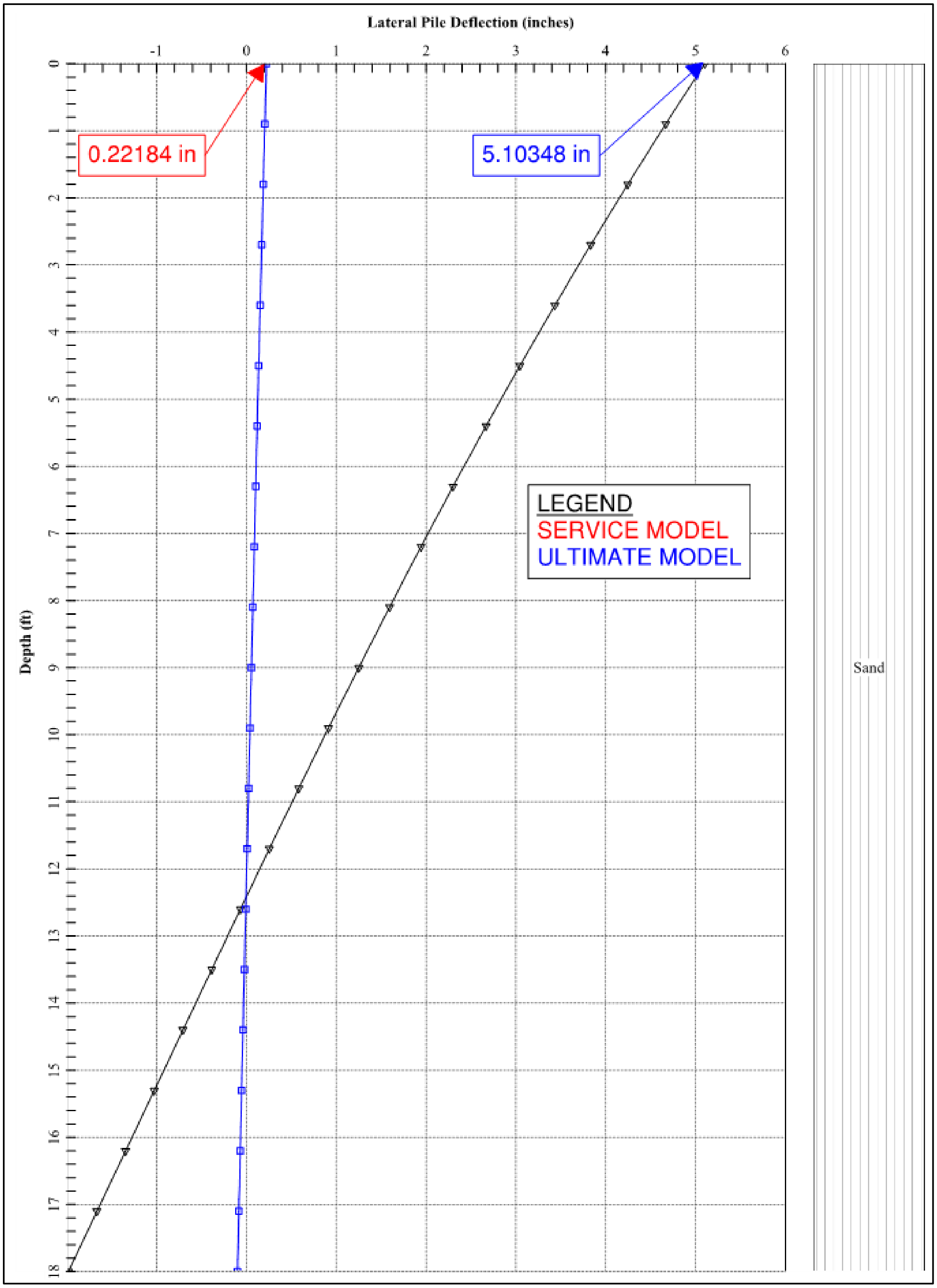


Figure 9-3: RISA-3D Rotated Structure (Ultimate)

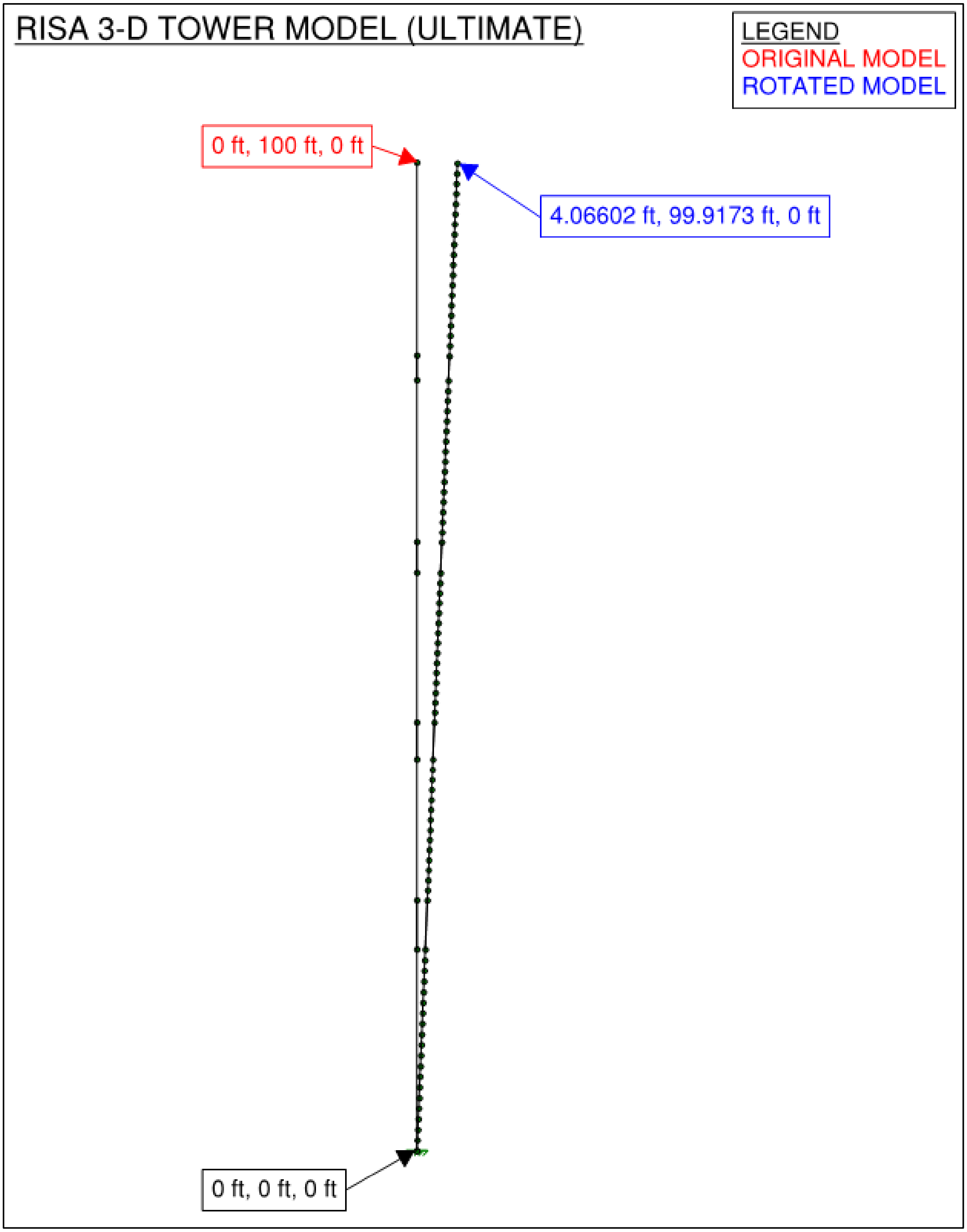
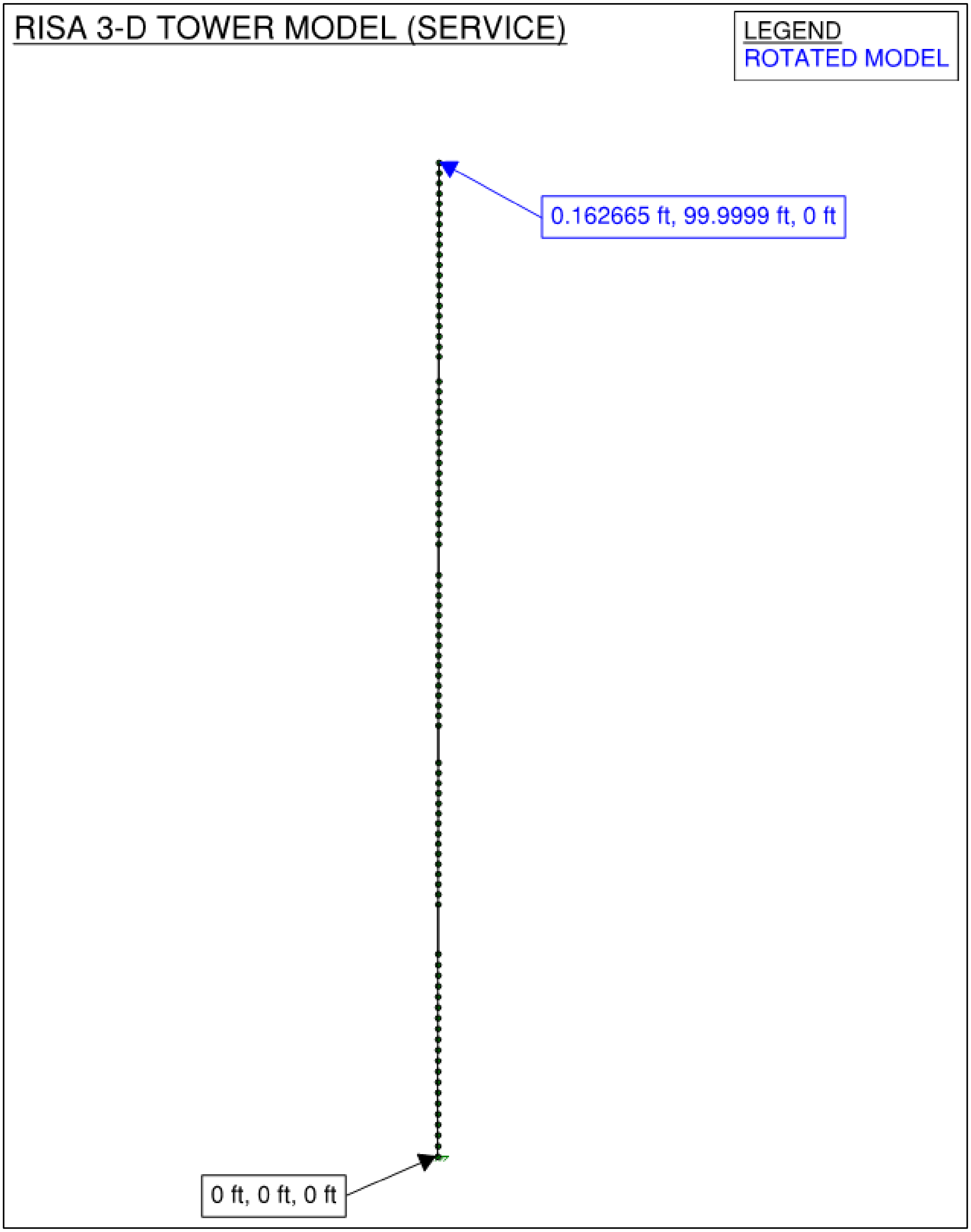


Figure 9-4: RISA-3D Rotated Structure (Service)



CHAPTER IV
ANLYSIS MEHTODOLOGY AND COMMENTARY OF RESULTS

Broms Analysis methodology:
The results of a Broms analysis for the same foundation are provided in Figure 9-1. The center of rotation was found to be at 12.96 ft. below grade with a corresponding applied moment, Mu, of 2319.97 k-ft. However, the total resisting moment is only 2020.23 k-ft. As a result, the Broms method analysis would indicate that the foundation fails at 114.8%.

Commentary of results:
Broms method does not provide any pile deflection results. Instead, it relies on simplifying assumptions to provide a capacity threshold for the resisting soil forces to ensure a safe design.

In contrast, the performance-based method allows the use of more sophisticated soil-structure interaction to take into account actual foundation and structure displacements. The engineer can then consider the resulting displacements and evaluate the impact on the supported structure. In this way, a safe analysis of the structure and foundation can be completed that more accurately captures the behavior of the system.

The comparison of results shows that there is potential to realize additional overturning capacity with a performance-based approach for a standard foundation type and soil conditions. The basis for acceptance is that the focus is more on the tolerable rotation and deflection limits for the structure and supported equipment. Due to the methodology, this will result in a pass/fail condition rather than a utilization capacity. The cross-section capacity checks, however, remain similar for each approach.

CONCLUSION

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 foundation reinforcement for telecommunication structures. Although traditional industry methods of analysis have historically resulted in safe and effective foundation designs, additional investment in engineering can provide an opportunity to optimize foundation capacity for key sites.

Recommendations for consideration include:

1. Utilizing performance-based design for deep foundation rather than Broms method in order to more accurately model the actual behavior of the foundation.
2. Eliminating the 1/phi (1.33) increase of factored reactions for performance-based analysis to maintain a more realistic representation of foundation and structural deflections.
3. Coordinating with the Geotechnical engineer to ensure that p-y curves used in the analysis are appropriate for the soil conditions at the site with adequate safety to prevent failure due to soil strength.

These analysis methodologies have been successfully used for foundation analysis and design in other industries. The IBC code and AASTHO standard 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.

AUTHORSHIP CONTRIBUTIONS

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