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26 October 2018

Mr. Tom C. Hui, S.E., C.B.O
Director
San Francisco Department of Building Inspection
1660 Mission Street
San Francisco, CA 94103

Via e-mail and US mail:

Re: Response to Letter Requesting Additional Information from the Millennium Tower Homeowners Association

Dear Mr. Hui:

As requested in your August 23, 2018 letter to Mr. Vision Winter, we are providing you with additional clarification regarding the basis of our opinion about the safety of the existing building foundation for the Millennium Tower at 301 Mission Street on behalf of the Millennium Tower Association. For clarity, we've repeated your questions in bold and followed with our answers.

1. ***Pile Stiffness and Settlement Analysis (Figures 13/14 and related text):*** **Please confirm the definition and basis of "long term" as related to the pile stiffness term KOLD BAY CLAY.**

   a. **Is the stiffness based on the expected settlement three years into the future (i.e. the expected time for completing a foundation retrofit as stated on Page 2) or the present state? How much additional settlement is predicted over the next three years?**

      The initial KOLD BAY CLAY stiffness value is calibrated to the settlement observed on April 12, 2018 (as noted in the caption of Figure 14 in the original letter).

The predicted additional settlement over the next three years is calculated by measuring the change between the first reading on 1/30/17 and the reading on 04/12/18 (437 days) – and then assuming each survey point continues to settle at the same observed rate for the next 3 years (1612 days) to 6/30/2021. See attached Figures 1 to 4 for the three-year settlement forecast and resulting change in pile head rotations.

b. Does using combined KPILE and KOLD BAY CLAY stiffness result in settlement observed to date? What values of KPILE and KOLD BAY CLAY have been used?

Yes, the analytical mat displacements shown in the previous report (Figure 14) were calculated using combined KPILE and KOLD BAY CLAY stiffnesses, as springs in series. KPILE is a nonlinear spring, representing the combined geotechnical and structural stiffness within the depth of the pile. These stiffness values vary by zone across the site. See Figure 5. KOLD BAY CLAY is a linear spring representing the long-term response of Old Bay Clay strata to sustained loads. The values vary across the site. See response to Question 1c below for a description of how these stiffnesses were calculated and a mapping of their values.

c. Confirm how the pile properties have been calculated (and differentiated across the building site) to result in calculated gravity load settlement displacements in the ETABS analysis.

In order to provide vertical springs to model the existing piles, ENGEO developed empirically based geotechnical capacities for each pile in combination with the information gathered during construction (i.e initial driving blow counts, PDA testing and wave analysis on a number of piles). The mat then was divided into zones of similar pile capacities. In addition, EGNEO added the elastic shortening of each pile to create a complete vertical spring at each pile location.

The KOLD BAY CLAY spring stiffness values were initially calculated as the elastic stiffness (EA/L) of a column of soil, with the following parameters:

- E = the long-term static stiffness of the soil, taken as 10 ksi (median value for hard clay, per Bowles T2-8);
- A = area of soil tributary to a pile; and
- L = soil column height, calculated as the distance from the base of the pile tip to the top of the bedrock (see Figures 6 & 7)

These initial spring stiffnesses were modified to account for other effects of the site configuration. For example, we have assumed increased stiffness at the perimeter to account for the stiffness contribution from adjacent soils (see Figure 8). We then increased the pile stiffnesses in the zone nearest the mid-rise structure due to the unloading of soil stress as a result of excavation for the parking garage (see Figure 9).

We then iterated on the initially calculated stiffnesses, adjusting stiffnesses to more closely match the surveyed mat deformed shape. See Figure 10 for a mapping of the adjusted long-term pile stiffness values. The result is a very close match, typically within 3%, with measured mat deformation and settlement.

d. Are the pile stiffnesses used in the nonlinear dynamic analysis based on present settlements or the estimated future settlements.

The pile vertical stiffness used in the nonlinear time history analysis is the dynamic stiffness, or KPILE, which isn't affected by time. For the pile lateral stiffness in the nonlinear time history analysis we have considered both settlements today and in three years. There is only a modest difference in the stiffness between today and three years from today.

2. *Pile Stiffness:* Does the plot in Figure 15 refer to axial or lateral foundation stiffness and strength. Is this a notional plot or does it represent the actual type of model (bilinear) used in the analyses?

This plot is conceptual. It represents either lateral or vertical stiffness and is meant to illustrate the concept of geotechnical bounding and the efforts taken by the LERA/ENGEO team to tighten the bounds recommended by ASCE 41.

3. ***Assumed Site Class (Figure 16):*** **Considering that the point of pile fixity and the zone of significant soil-structure interaction is within Young Bay mud, please justify using Site Class D for establishing the input ground motions.**

The following response is provided by Uri Eliahu, President of ENGEO Incorporated:

Based on our experience with similar construction, soil-structure-interaction, depth of basement, and number of piles, it is our opinion that current site is should be classified as Site Class D. We have been performing on-going evaluations to demonstrate this analytically.

In order to confirm Site Class D designation, the following evaluation is ongoing:

- *Create a 2-Dimensional finite-element model in Plaxis of the subsurface conditions under the building, including the concrete piles*
- *Evaluate the site response at the surface with relatively small earthquakes to keep soils within the linear range.*
- *The first evaluation includes the concrete piles with appropriate elastic response*
- *The second evaluation does not include the piles, but only the soils with the appropriate elastic response.*
- *Compare the surface spectra for both evaluations*
- *Increase the stiffness of the soils until the second evaluation matches the first.*
- *Determine the equivalent shear wave velocity of the soils after the spectra are matched.*

Early indications are that these steps will confirm that the site as is can be classified as a site class D on an analytical basis.

4. ***Pile Response (Figures 21 & 22 and related text):*** **Related to the pile lateral load deflection curves shown in Figure 21 & 22, please:**

a. **Confirm if L-PILE was used to develop the pile lateral load curves and whether these curves represent the response of a single pile or a group of piles.**

Confirmed, L-PILE was used. These curves represent the response of single piles, with factors applied to account for the group effect of closely spaced piles. ENGEO developed the group effects using the Reese's numerical methods (same algorithm used in the software Group by Ensoft). The evaluation consisted of subdividing the mat into smaller groups of piles and obtaining an average reduction of lateral resistance on the transvers and longitudinal directions. The average reduction for a typical pile resulted in a p-multiplier of 0.65 by averaging transverse and longitudinal directions.

b. **Explain the underlying mechanisms that result in the strain hardening behavior in these curves. Is there any available experimental data in the published literature to substantiate this type of strain hardening behavior out to large deflections?**

The strain hardening observed in the pile lateral load-deflection curves is due to the strain hardening characteristics of the materials that comprise the piles. These material properties are illustrated in Figure 11, which includes their references. These material behaviors are well understood and well documented.

c. **Provide a diagram of the deformed shape of a representative pile as it undergoes lateral deflection, including the locations of hinge formation.**

We have observed two types of hinge formation sequences:

i. Case 1: A hinge forms at the pile connection to the mat. This hinge then fails prior to the formation of the second hinge. See Figure 12.
ii. Case 2: A hinge forms at the pile connection to the mat. Then, a second hinge forms at some distance below the top of pile prior to the top hinge failure. See Figure 13.

Note that hinge failure is defined as exceeding the strain limit of either the confined concrete or pile reinforcement.

d. **Confirm the reinforcing bar layout that is assumed in the pile analyses at the hinge locations.**

   See Figure 14.

e. **Confirm whether the shear capacity of the piles has been checked to ensure that shear failure does not occur before hinge formation in the piles.**

   Confirmed, the pile structural shear capacity exceeds the maximum lateral force resistance for all values of axial load. See Figures 15 & 16.

5. ***Upper vs. Lower Bound Soil Properties (Figure 22):*** **Are the differences between the upper median and lower bound plots in Figure 22 representative of the variation in response for all of the piles? Can you provide analysis results to show how much the dynamic response will vary between the lower, median and upper bound cases?**

Yes, the variation shown in Figure 22 from the previous report was a representative example.

Yes. The results of lower, median, and upper bound were described in the text of the previous letter dated July 30, 2018. Please refer to the third paragraph of our answer in question 4.

6. ***Unbalanced Active Earth Pressure and Soil Stiffness (Figures 25 & 26 and related modeling assumptions):*** **Related to the modeling of unbalanced active earth pressure and soil stiffness assumed on the west and north basement walls, please:**

a. **Provide a description and/or diagram to illustrate how the foundation and surrounding soil are represented in the nonlinear structural analysis model (ETABS) and where the ground motions are introduced into the model.**

   Passive pressure is modeled using compression only nonlinear springs. See Figure 17.

b. Explain how the unbalanced active and seismic soil pressure is combined with seismic ground shaking.

Active and seismic earth pressures are applied as initial static loads to the west and north foundation wall.

c. Explain the meaning of the three lines shown in the pile pushover plots (dashed line and two solid lines).

The dashed line represents passive pressure, and the thin solid line represents the existing pile resistance to lateral loads. The thick solid line represents the combination of the two.

d. Explain why when pushing south, the first pile yield occurs at zero displacement.

A 0.5 degree rotation at the top of an existing pile causes yielding in the pile reinforcement just from the rotation. Since the non-uniform settlement and tilting has already caused a few of the piles in the south direction to experience 0.5 degrees of rotation these piles are presently in a yielded state prior to the start of any seismic activity. Refer to Figures 2 & 3 for a mapping of pile head rotations.

7. ***Nonlinear Foundation Modeling (Figure 20 and related text)*:**

a. Please clarify the difference between the model used for the global foundation pushover, and nonlinear time history analysis. Figure 20 suggests piles may have been modeled differently for both types of analysis.

There is no difference. The same lateral pile force vs. deflection models described in the previous letter were used for both the global foundation pushover plots and the nonlinear time history analysis. Each pile was modeled individually. The global foundation pushover is simply the summation of the individual pile backbone curves used in the nonlinear time history analysis. The individual pile backbone curves are based on each pile's unique axial load and initial rotation about each orthogonal axis.

b. **The last paragraph in the response to Question 4 suggests that interaction between vertical and lateral response may not have been explicitly modeled when determining the "foundations global backbone curve". Is this also true for the nonlinear dynamic analyses (see question 7a above)? It is unclear why this would only have an effect for loading in the south and east direction (as stated in the letter).**

As noted in 7a above, the lateral backbone curves for each individual pile were selected based on the axial load in that pile due to gravity. As indicated in the last paragraph of our response to question 4 dated July 30, 2018 this is a somewhat optimistic view of foundation capacity as it does not account for the force amplification that occurs in piles during a seismic event.

Also, as noted in 7a above, we have modeled the individual pile backbones. The global backbone is simply a summation of the individual pile backbones. Hence, the global backbone is not used in the nonlinear time history analysis, but rather is provided for reference only.

The southern and eastern directions are not the only directions affected, but given the directional nature of the pile damage due to differential settlement and tilt, the south and east directions are more interesting relative to the question of foundation non-convergences.

8. ***Nonlinear Dynamic Analyses (Figures 25 & 26 and related text):* To help substantiate the nonlinear dynamic analysis results under BSE-I and BSE-2 motions, please:**

a. **Confirm the clearance between the mat/basement walls and the structures to the east and south. Have your analyses considered the resistance provided by the adjacent structures and the possible impact between the tower mat and the adjacent structures (considering —5 inches of calculated displacement under MCE level ground motions)?**

The gap between the tower mat and adjacent parking structure to the east was noted as 1” in the original design drawing but was observed to be two inches when

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cores through the garage wall were reviewed. Regarding the Transbay Terminal to the south, there is 5 feet of soil between the tower's southern foundation wall and the Transbay foundation wall.

No, our analysis has not considered resistance provided by adjacent structures. The tower and adjacent low-rise structures to the east and south are independent structures, with very different periods of oscillation and separated by an expansion joint. We do not view the adjacent, independent low-rise structures as reliable paths for seismic load resistance.

We did consider the possibility of detrimental effects associated with impacts at the foundation level and concluded, given the strength and size of the 10' thick mat, that impact is not concerning to the performance of the tower mat foundation nor do we view the possibility of impact as having a detrimental effect on the piles.

b. **Please confirm your impressions of how to interpret the results of analyses that do not converge.**

   Our interpretation is that an analytical non-convergence represents a structural failure of the pile foundation system.

c. **Rerun and report results from nonlinear dynamic response under BSE-2 ground motions where the active earth pressure and unbalanced soil stiffness are ignored (i.e., ignoring the soil above the base of the mat on all sides of the building). This is important to provide a more direct comparison with the SGH analyses.**

   See Figure 18. We found 4 non-convergences out of the 11 ground motions when earth pressures were ignored.

9. ***Nonlinear Dynamic Analyses (Figure 27 and related text):* Please explain the horizontal and vertical mat displacement time history, as shown in Figure 27:**

   a. **As shown in Figure 27, the mat will have small lateral movement in the X-direction until about 9 seconds, when the displacement jumps to 3 inches, but then there are no oscillatory lateral movements beyond 9 seconds. In contrast, it appears the vertical displacement has oscillations after 12 seconds with a period of about 5 seconds (period of the structure?). Please explain the underlying horizontal and vertical mechanisms that can lead to the differences in the horizontal and vertical response histories.**

      The apparent difference is due to the different scaling of the graphs. When the displacement plots are evaluated at the same scale, we find that the lateral movements have oscillations similar to vertical displacement. As you note, the period of these oscillations is about 5 seconds which is close to the periods of the 1st and 2nd mode of the structure - 4.67 and 4.53 seconds respectively. See Figure 19.

   b. **Provide hysteretic response plots of the horizontal force-displacement and vertical force- displacement for one or more representative piles. Also, clarify how unloading response is modeled in the piles.**

      For both horizontal and vertical resistance, the piles are modeled as Multilinear Plastic links with isotropic hysteresis type in ETABS. Unloading occurs along a path parallel to the initial elastic stiffness.

      See Figure 20 for representative hysteretic response plots for both horizontal and vertical motions.

   c. **Confirm whether residual building drift will result from the vertical mat displacements in the BSE 1 and BSE 2 ground shaking.**

      Yes, residual drift will result. See Figures 21 & 22.

d. Confirm whether vertical ground motions are included in the analysis and their significance on the vertical response.

As is consistent with the evaluation of towers where there are no discontinuities in the vertical-load-carrying elements, vertical ground motions were not explicitly considered in the analysis.

Very truly yours,

Daniel A. Sesil
LERA CONSULTING STRUCTURAL ENGINEERS, RLLP

DAS/jbs
Enclosure

cc: Rick Riley, Riley Pasek Canty LLP, rriley@rileypasek.com
Charles Litt, Fenton Grant Mayfield Kaneda & Litt, LLP, charleslitt@fentongrant.com

APPENDIX A: FIGURES AND PLOTS

2018-04-12: **2021-06-30:**

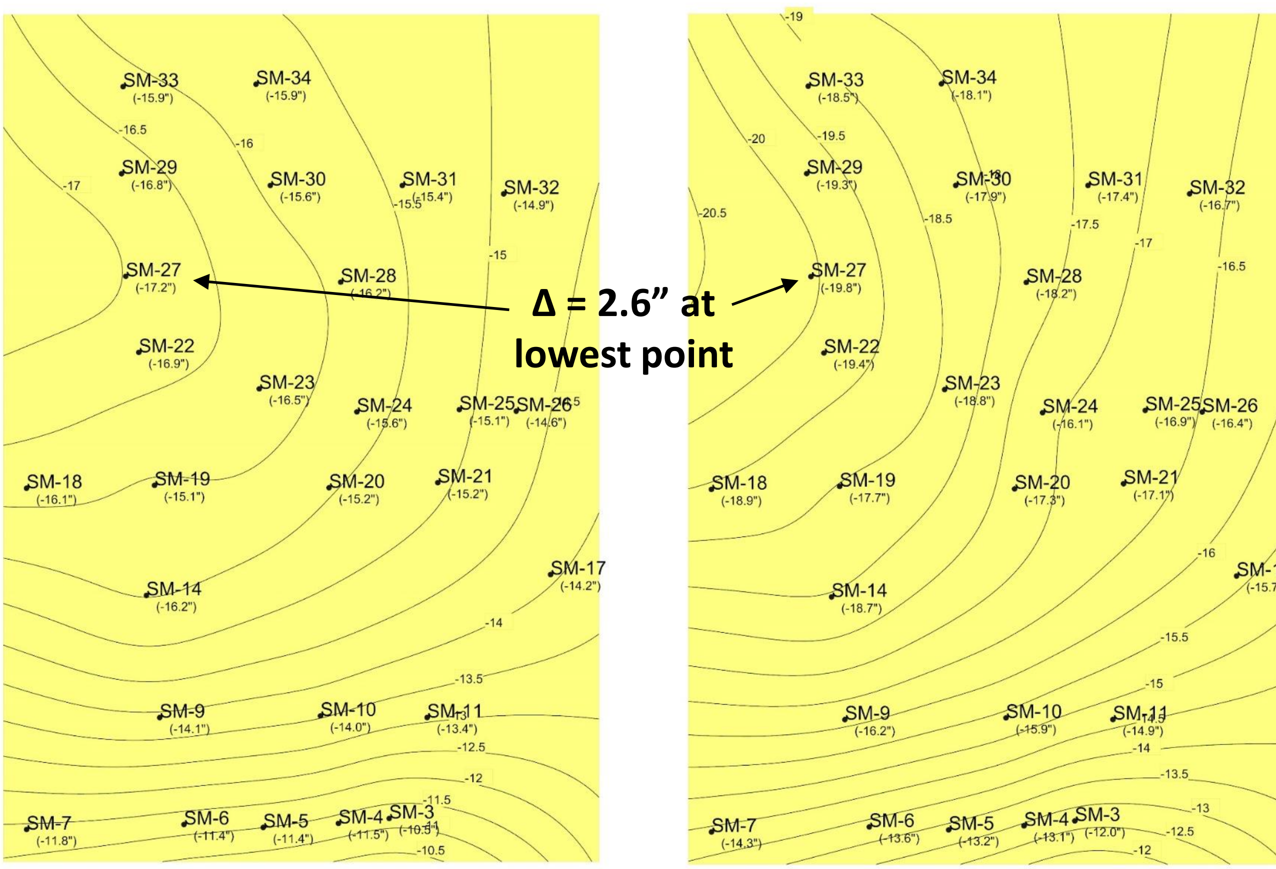


Figure 1. 3 Year Look Ahead Based on Recent Settlement Trends – Mat Displacement from Theoretical Elevation

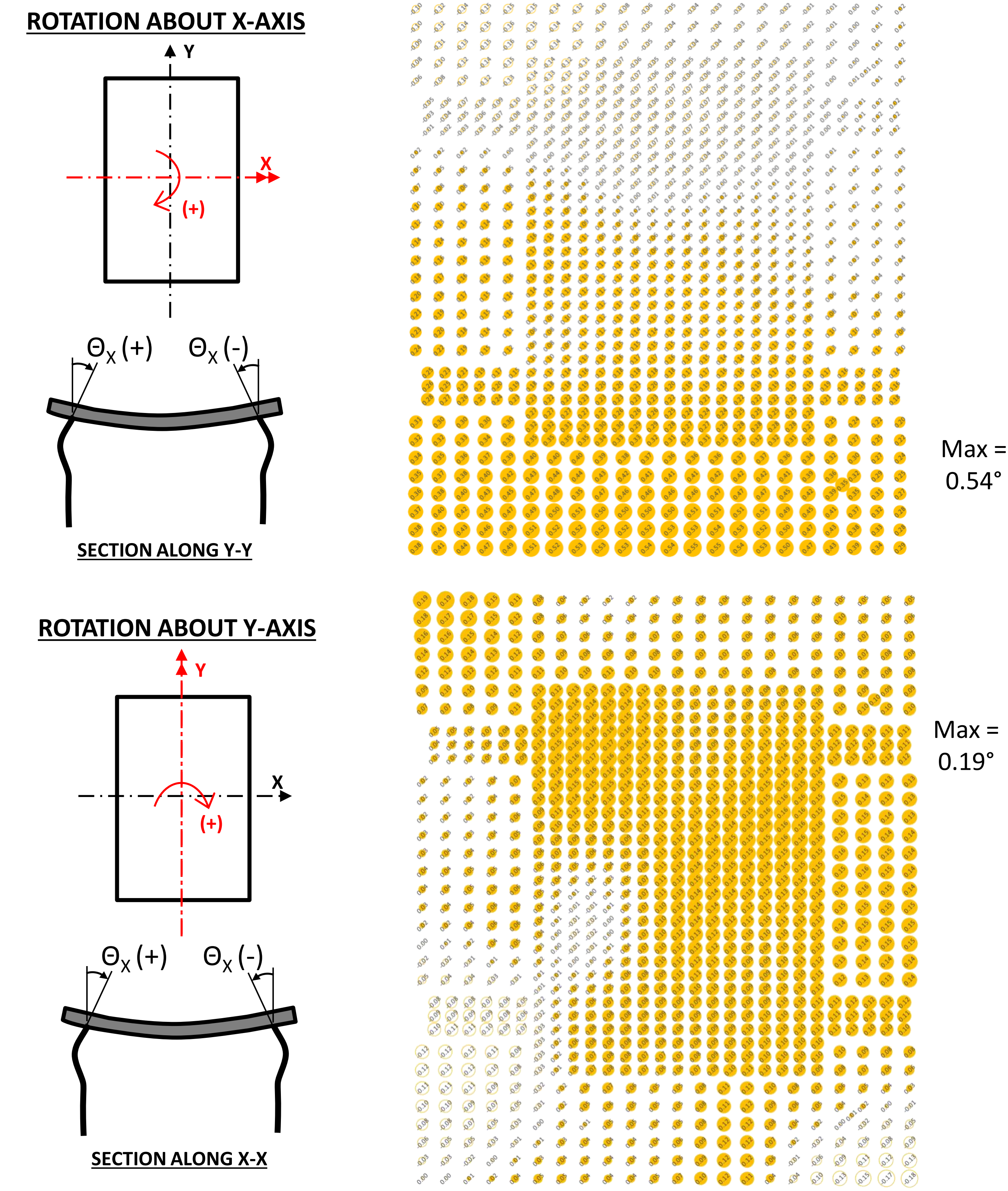
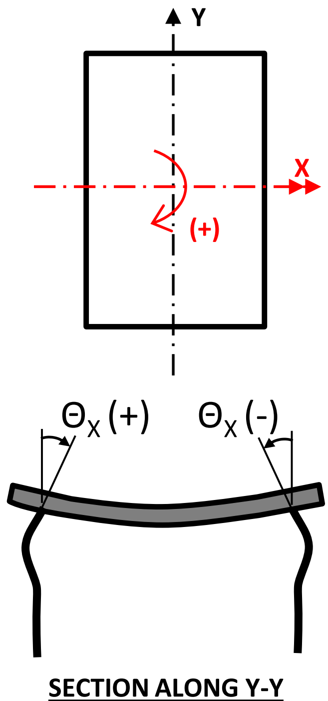


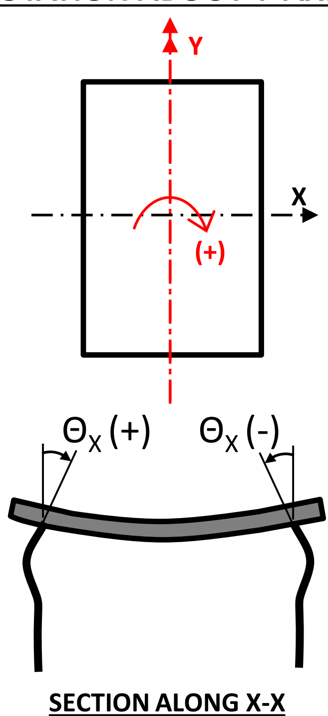
Figure 2. Initial Pile Head Rotations – (April 12, 2018)

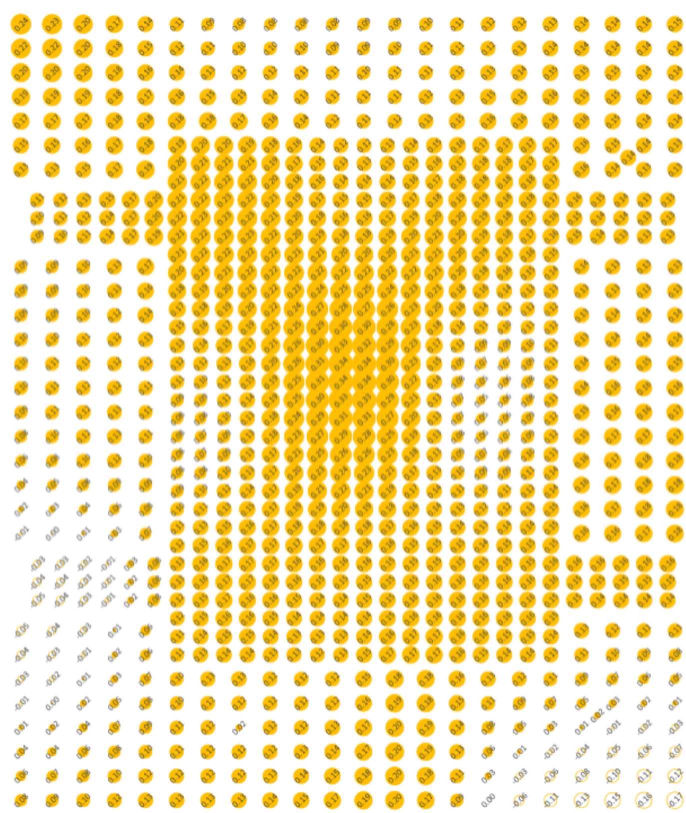
ROTATION ABOUT X-AXIS



Max = 0.60°

ROTATION ABOUT Y-AXIS

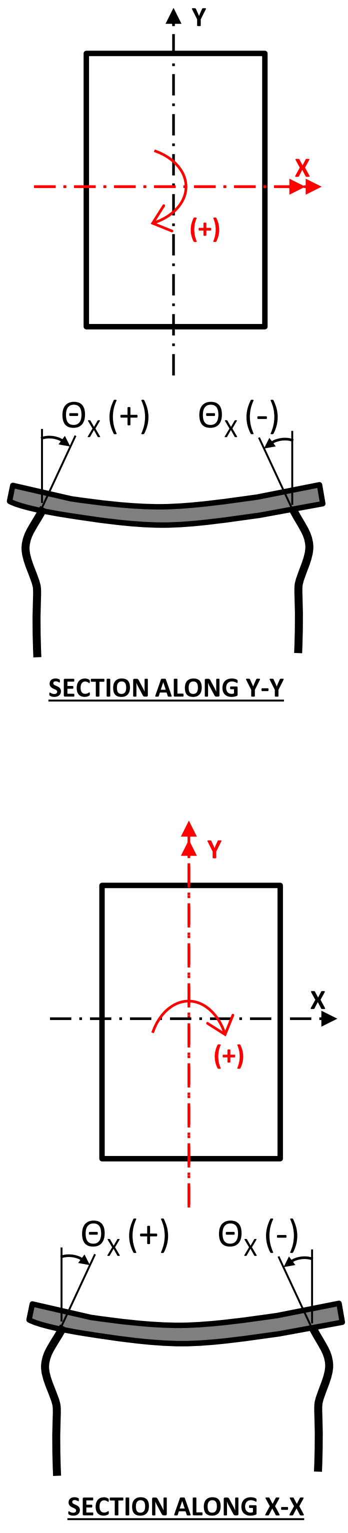




Max = 0.35°

Figure 3. Pile Head Rotations in 3 Years – (June 30, 2021)

ROTATION ABOUT X-AXIS



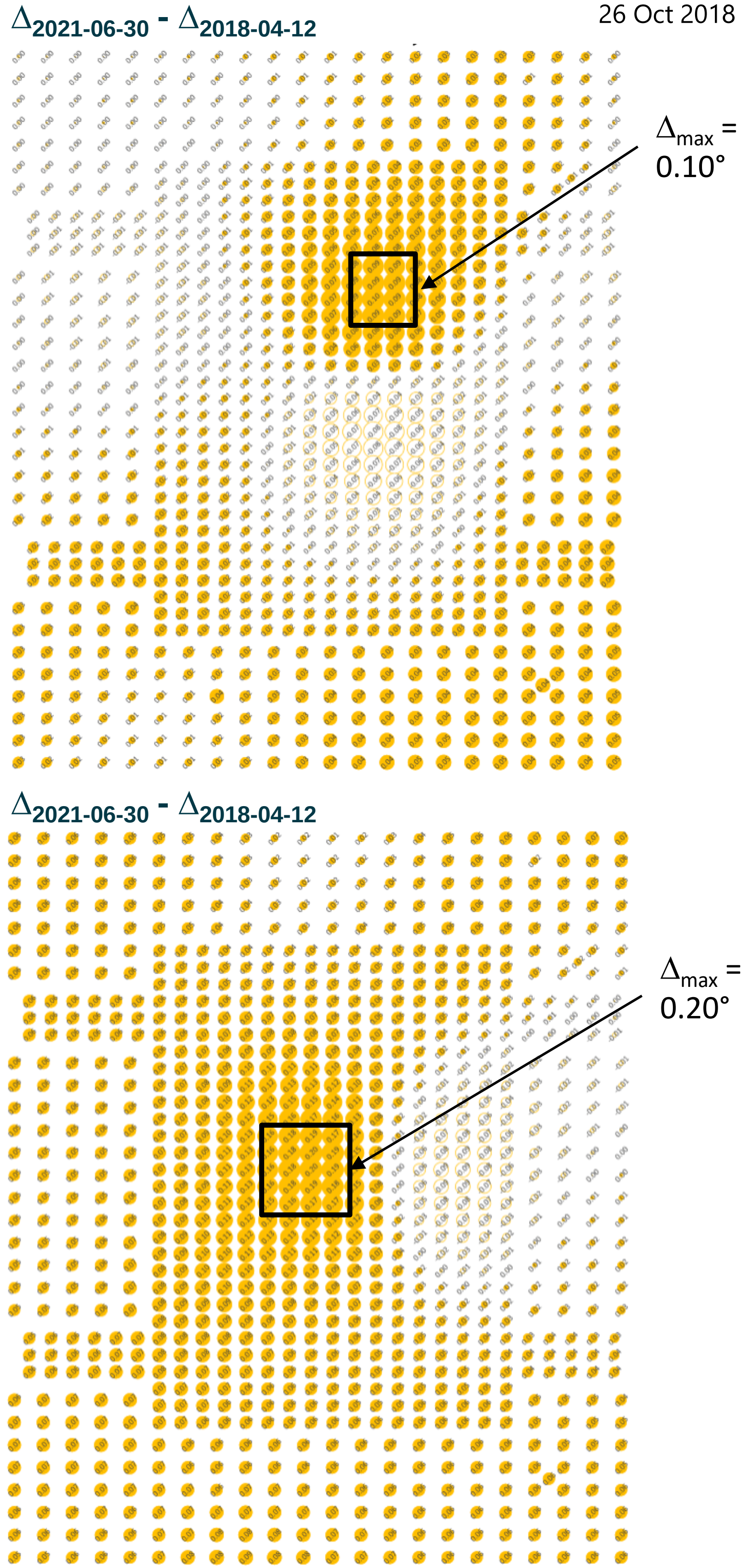
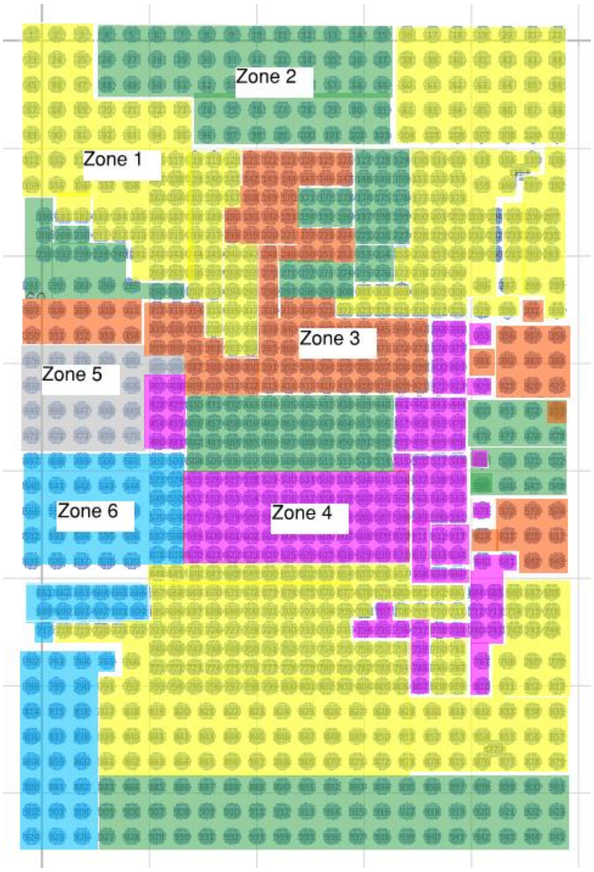


Figure 4. 3 Year Look Ahead Based on Recent Settlement Trends – Change in Pile Head Rotation



Existing Piles

ZONE	Ultimate Vertical Load Capacity					
	Compression			Tension		
	LB	AVG	UB	LB	AVG	UB
1	698k	775k	853k	220k	310k	350k
2	600k	667k	734k			
3	454k	505k	555k			
4	737k	819k	901k			
5	511k	567k	624k			
6	637k	707k	778k			

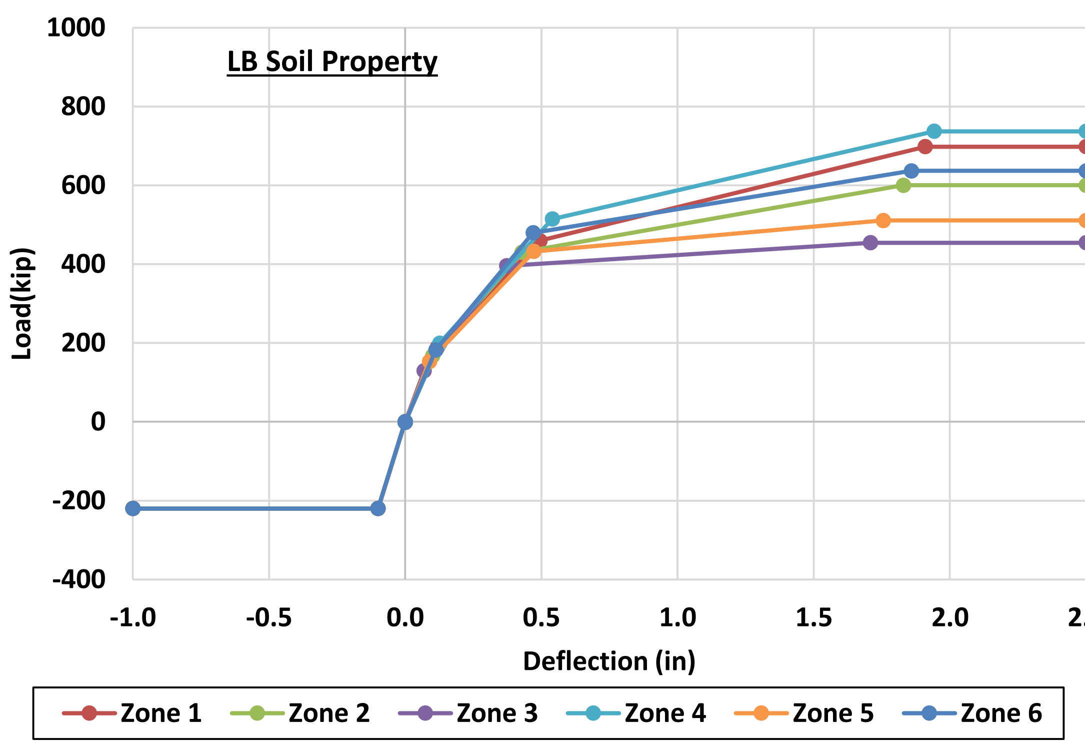
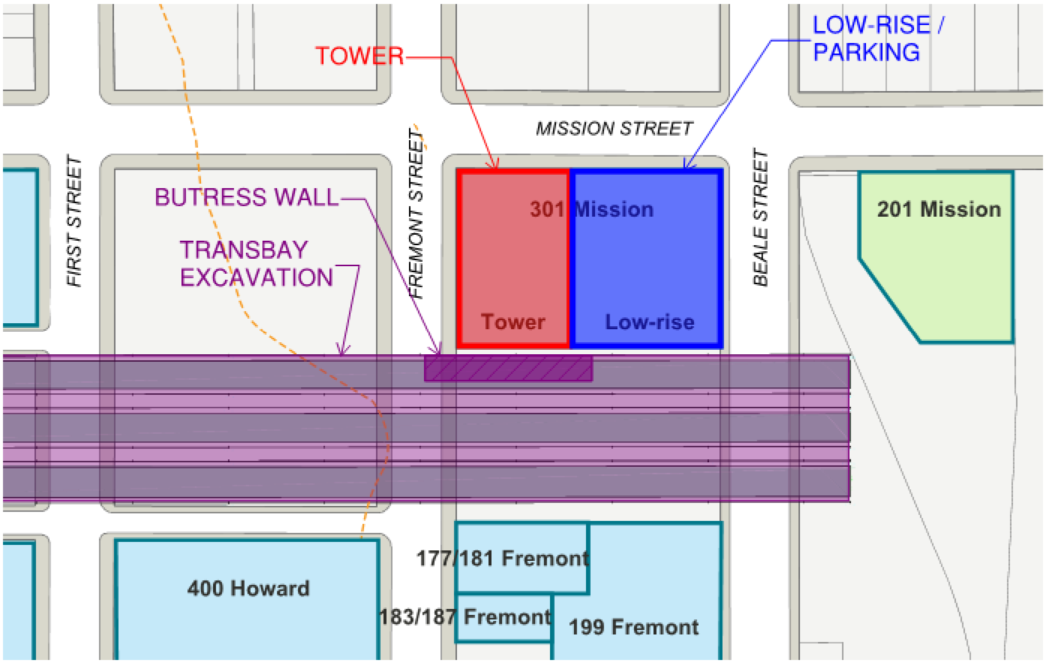
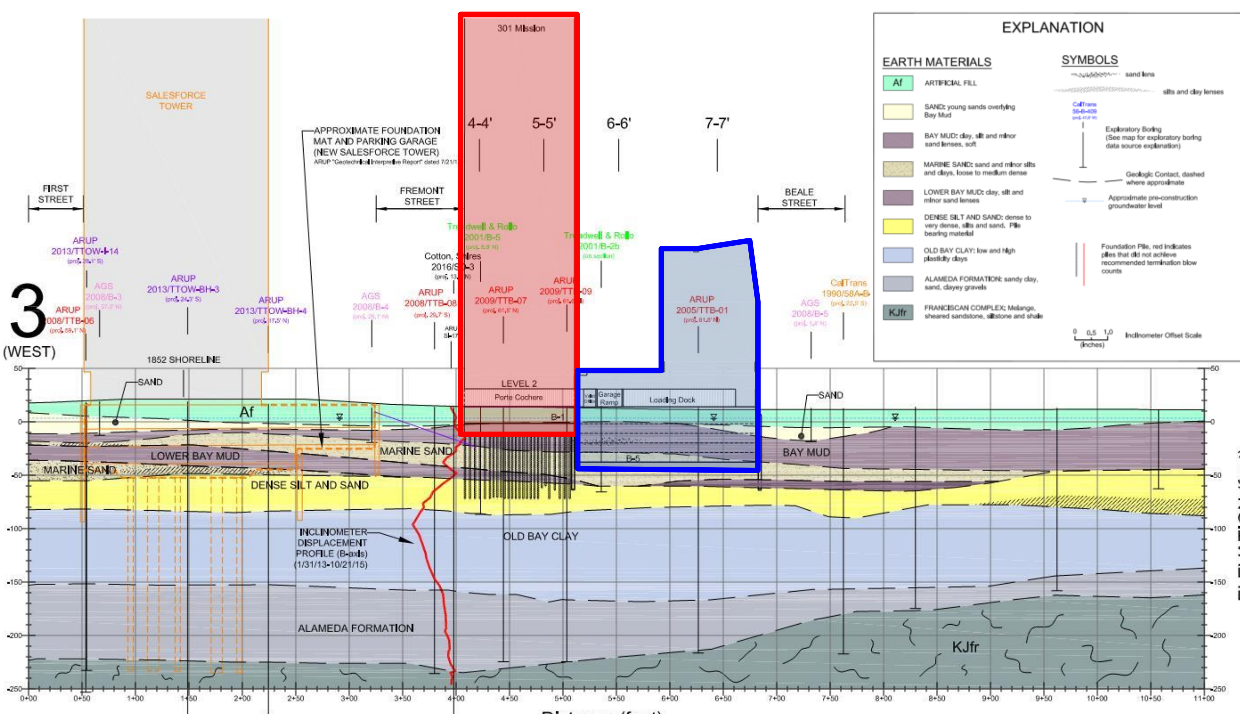


Figure 5. Existing Pile Vertical Stiffness / Capacity – Zones

Site Plan:



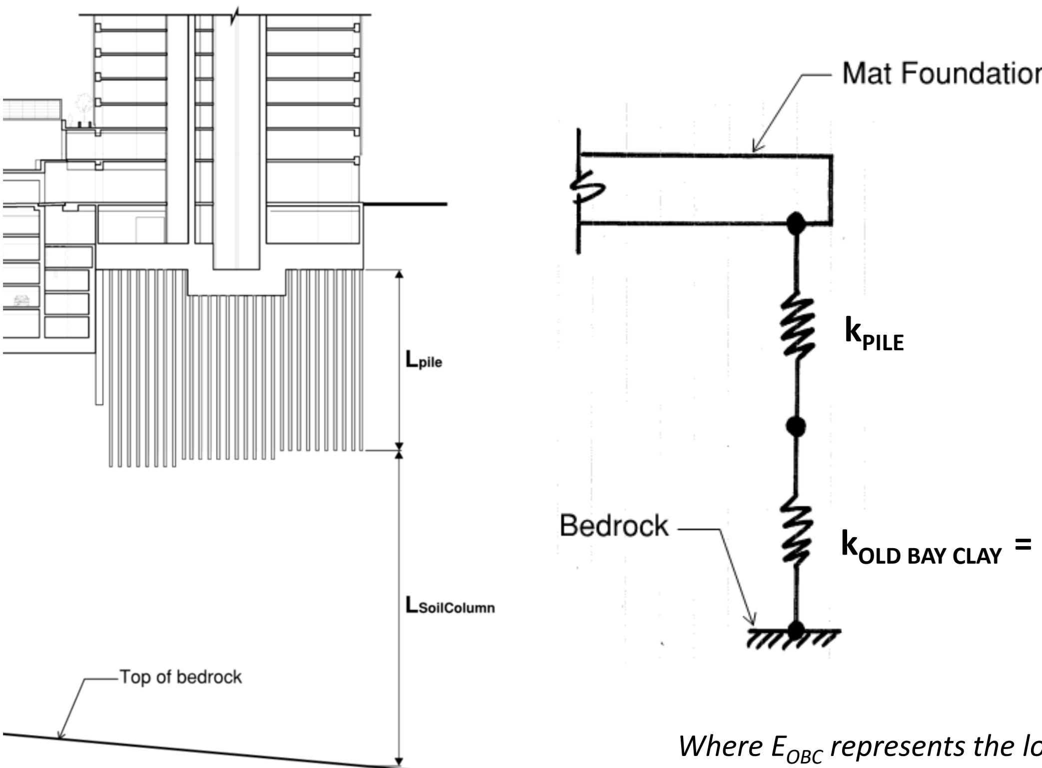
Section Looking North:



CSA Geologic Cross Section 3-3 (Dated February 2017)

Figure 6. Site Plan and Cross Section Through Strata to Bedrock

Effective stiffness of piles for gravity loads (springs in series):



$$k_{OLD\ BAY\ CLAY} = \frac{E_{OBC} * A_{TRIBUTARY}}{L_{SoilColumn}}$$

Where E_{OBC} represents the long-term static stiffness of the soil, taken as 10 ksi [median value for hard clay, per Bowles T2-8].

Figure 7. Pile Stiffness Calculation

We have assumed increased stiffness at the perimeter to account for the stiffness contribution from adjacent soils:

- $1.25 \times k_{OBC}$ = stiffness for outermost three layers
- $1.0 \times k_{OBC}$ = stiffness for interior piles

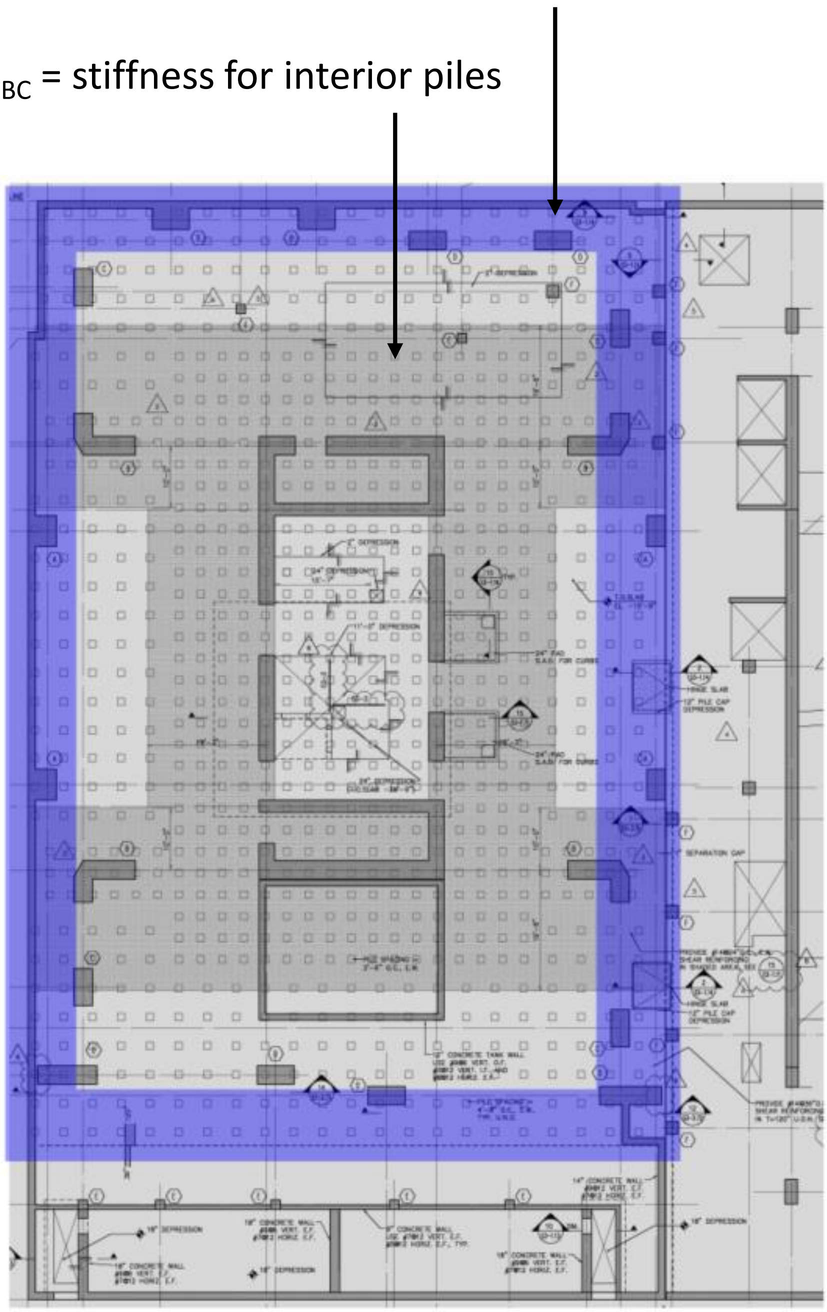


Figure 8. Pile Stiffness Considerations

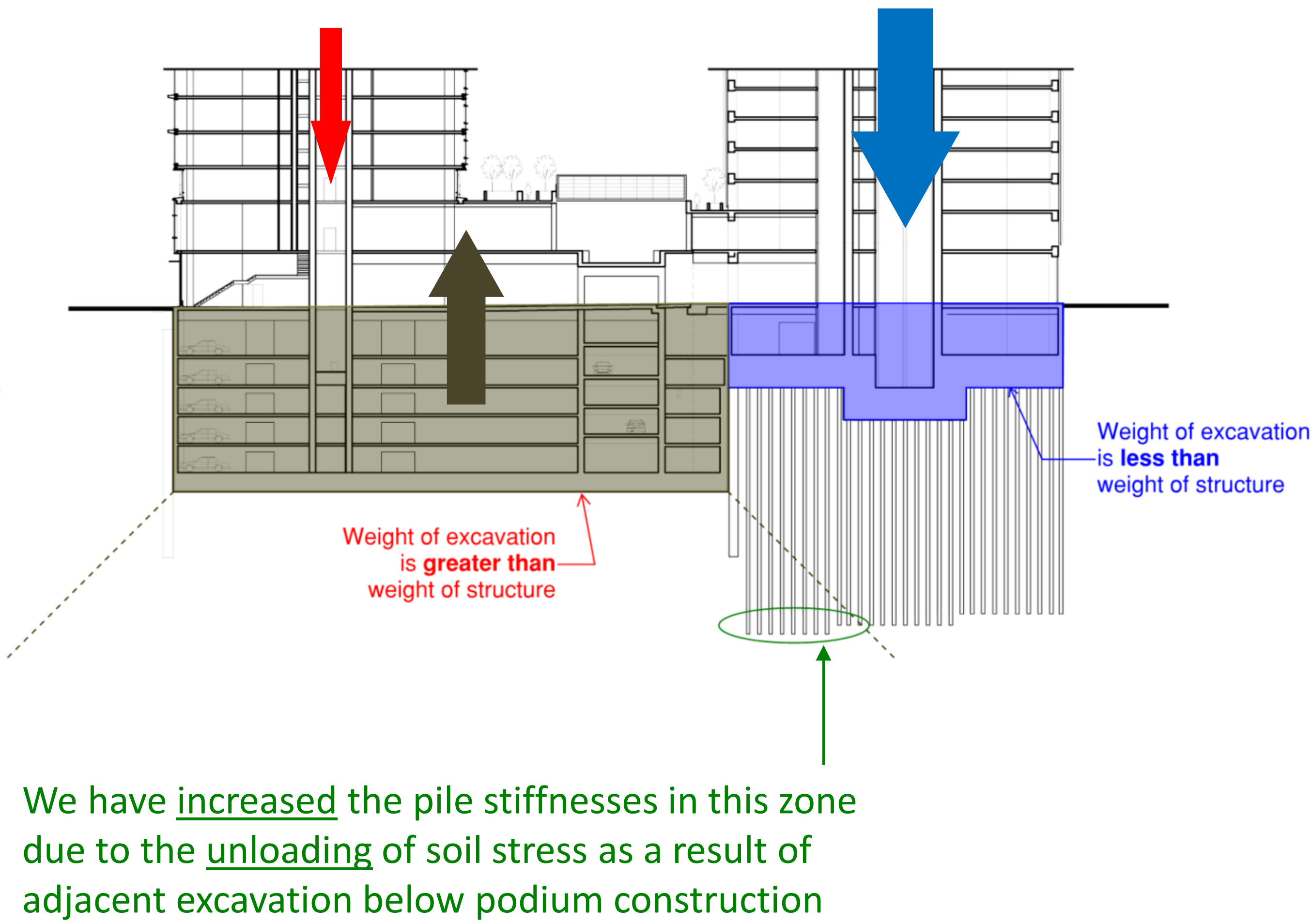


Figure 9. Pile Stiffness Considerations

MAPPED EFFECTIVE LONG-TERM PILE STIFFNESSES, k_{OBC} (k/in):

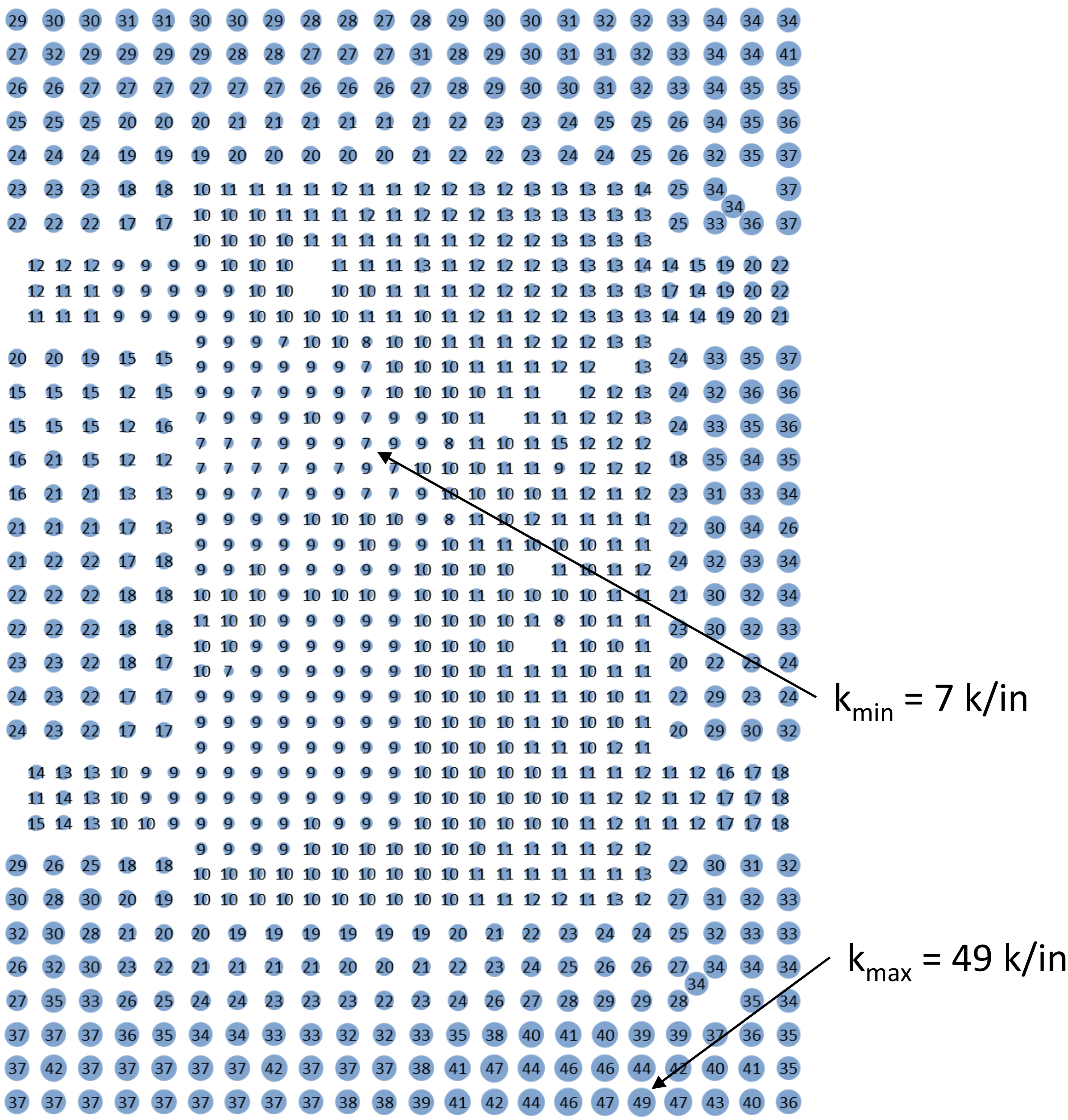
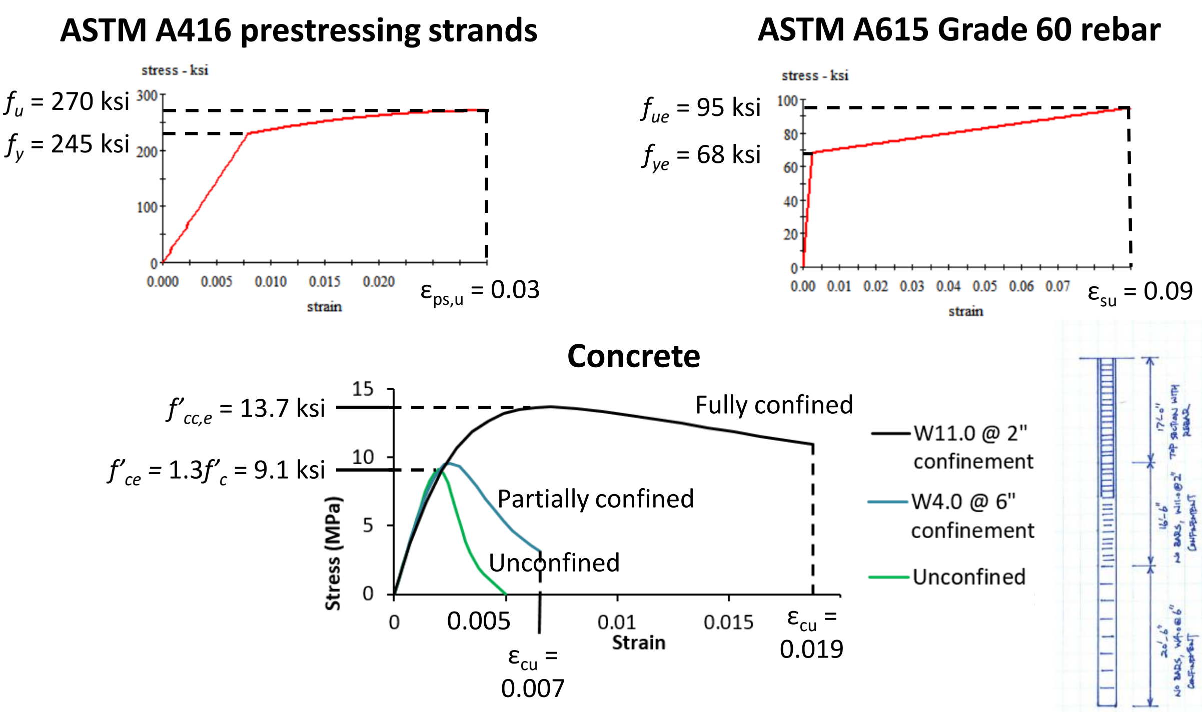


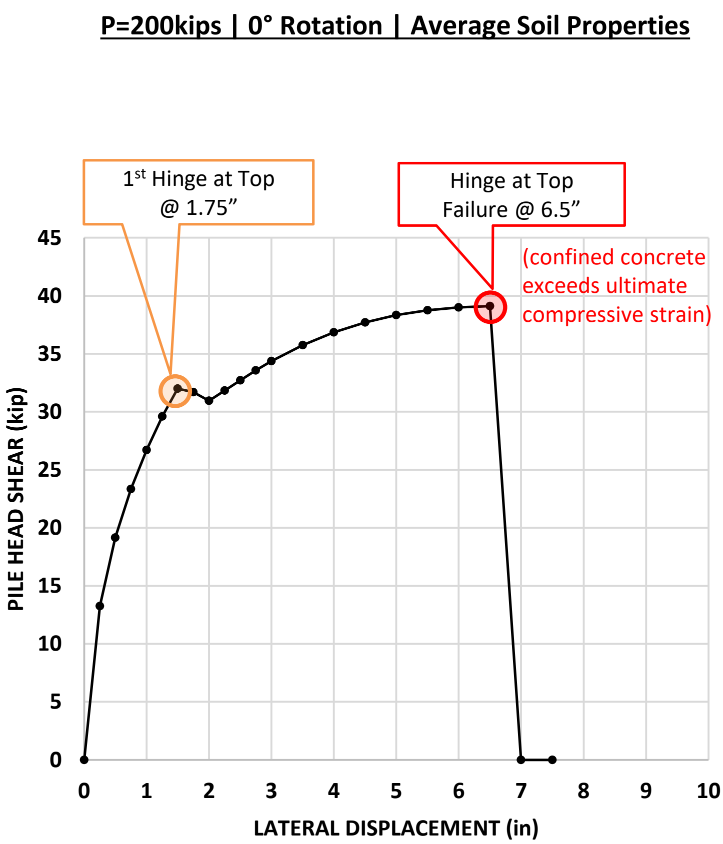
Figure 10. Long-term Pile Stiffness Accounting For Settlement



Reinforcement properties and strain limits determined in accordance with AASHTO *Guide Specifications for LRFD Seismic Bridge Design.* Concrete properties and strain limits calculated per Mander (1988) and *Seismic Design of Reinforced Concrete and Masonry Buildings* by Paulay and Priestley.

Figure 11. Material Models Used in XTRACT Pile Sectional Analysis

CASE 1



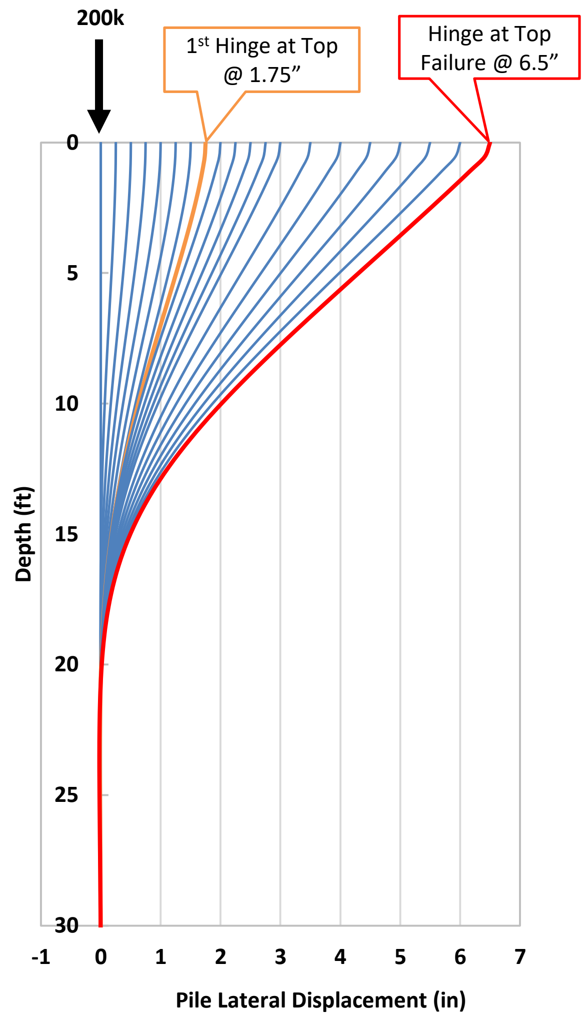


Figure 12. Pile Hinging Mechanism – Case 1

CASE 2

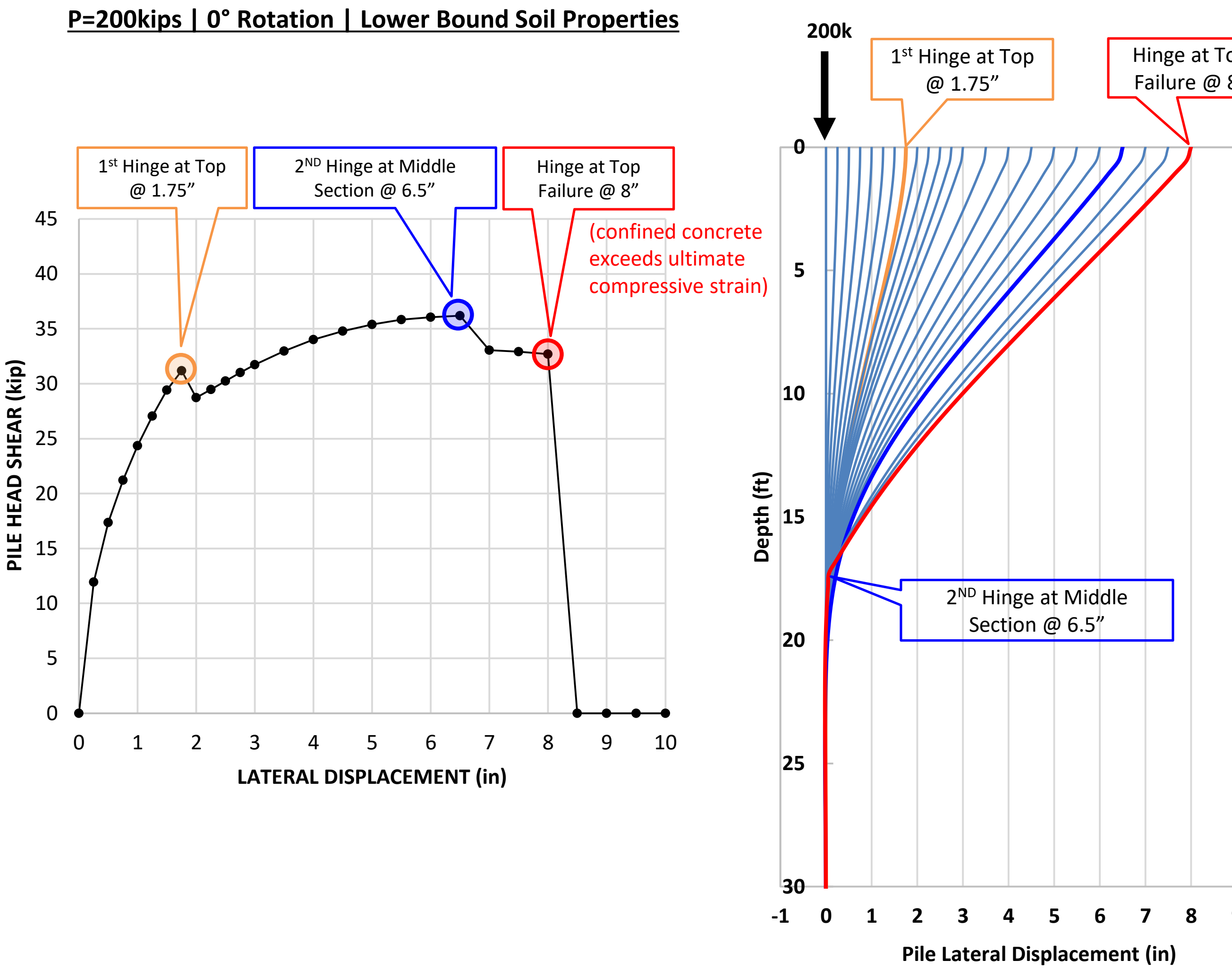


Figure 13. Pile Hinging Mechanism – Case 2

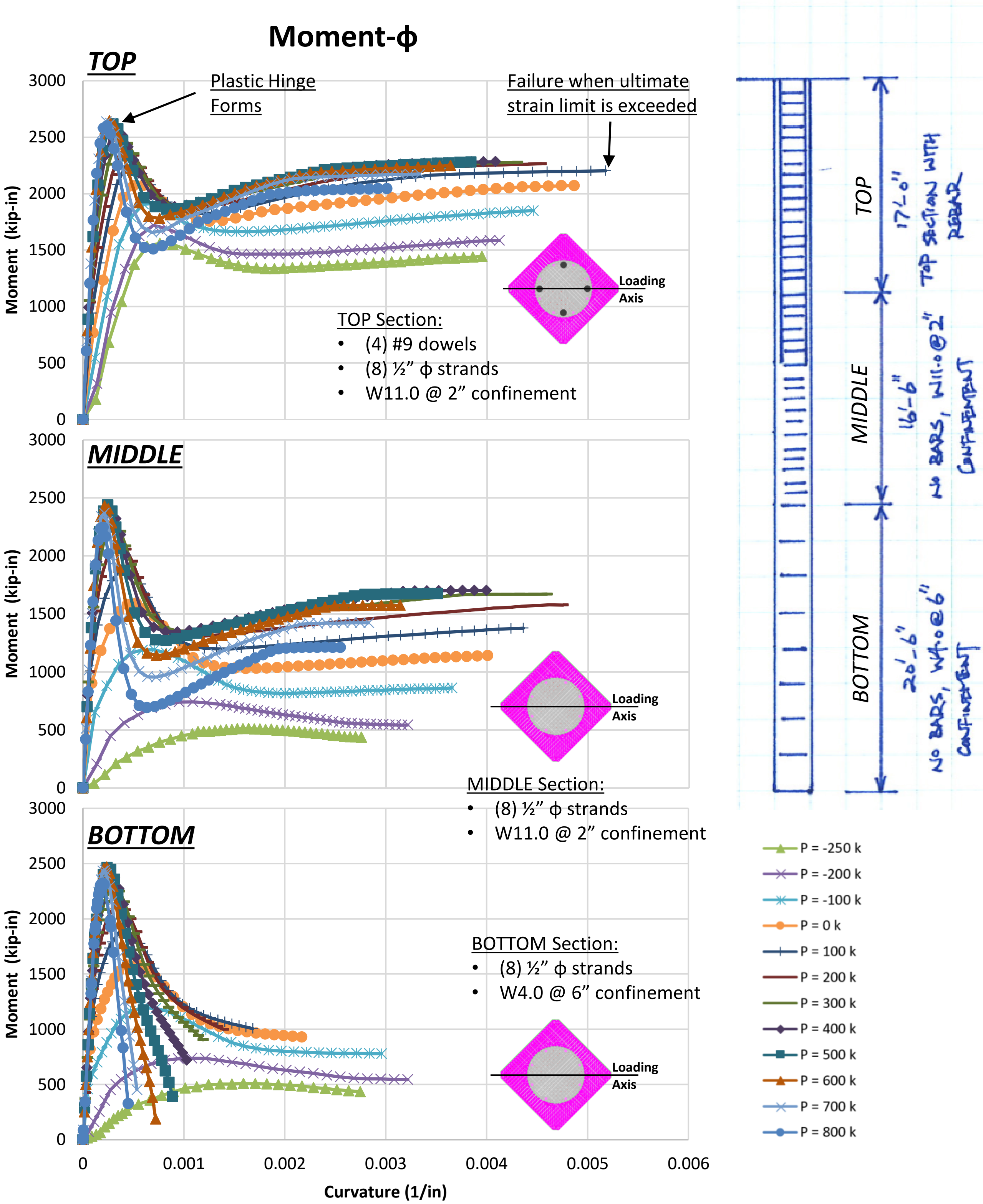
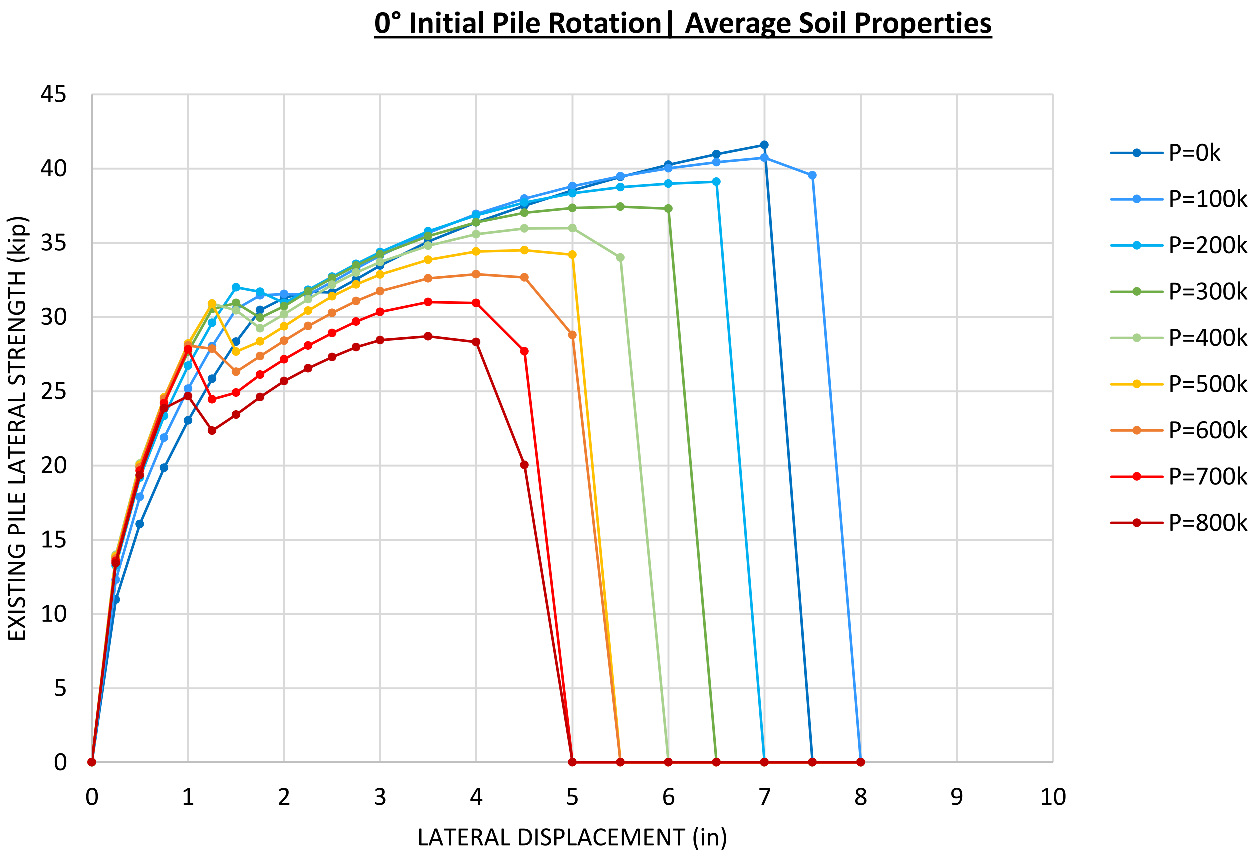


Figure 14. Typical Existing Pile Moment Curvature

Compressive Axial Loads:

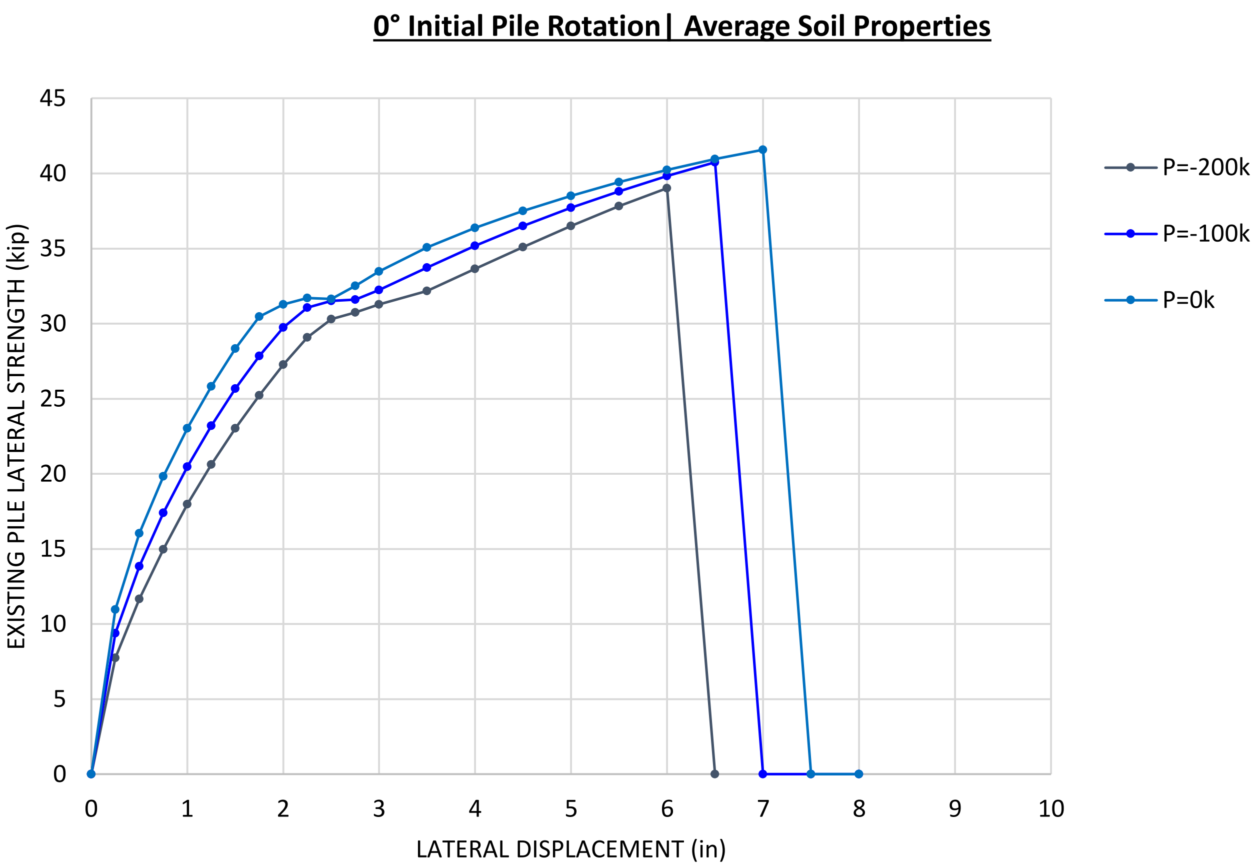


Note:
Structural shear strength of section exceeds the peak lateral shear for all values of axial load:

ϕVn (Pu = 0k) = 85k
ϕVn (Pu = 400k) = 105k
ϕVn (Pu = 800k) = 125k

Figure 15. Individual Pile Lateral Stiffness / Capacity – Compression Axial Load Variation with Shear Capacity

Tension Axial Loads:



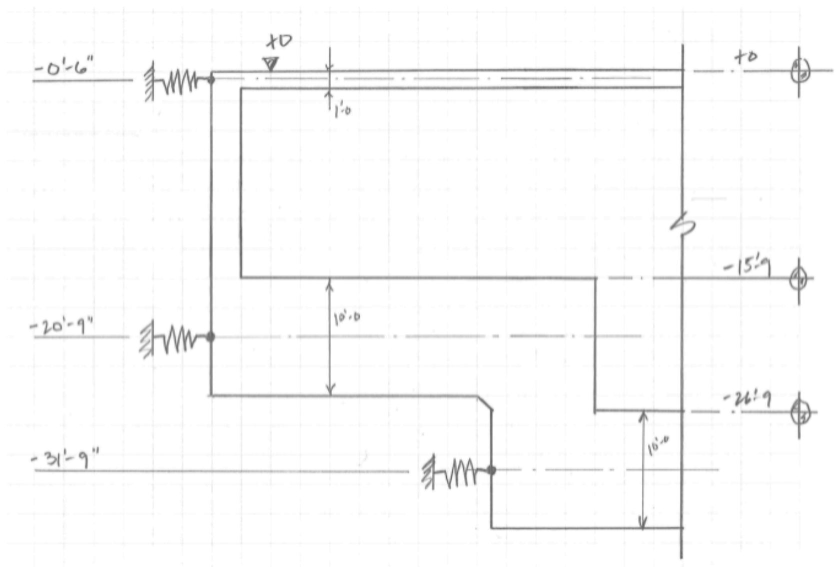
Note:
Structural shear strength of section exceeds the peak lateral shear for all values of axial load:

ϕVn (Pu = 0k) = 85k
ϕVn (Pu = -200k) = 65k

Figure 16. Individual Pile Lateral Stiffness / Capacity – Tension Axial Load Variation with Shear Capacity

Passive Earth Pressure:

Passive soil resistance to lateral loads is accounted for on west and north basement walls, and at the elevator pit



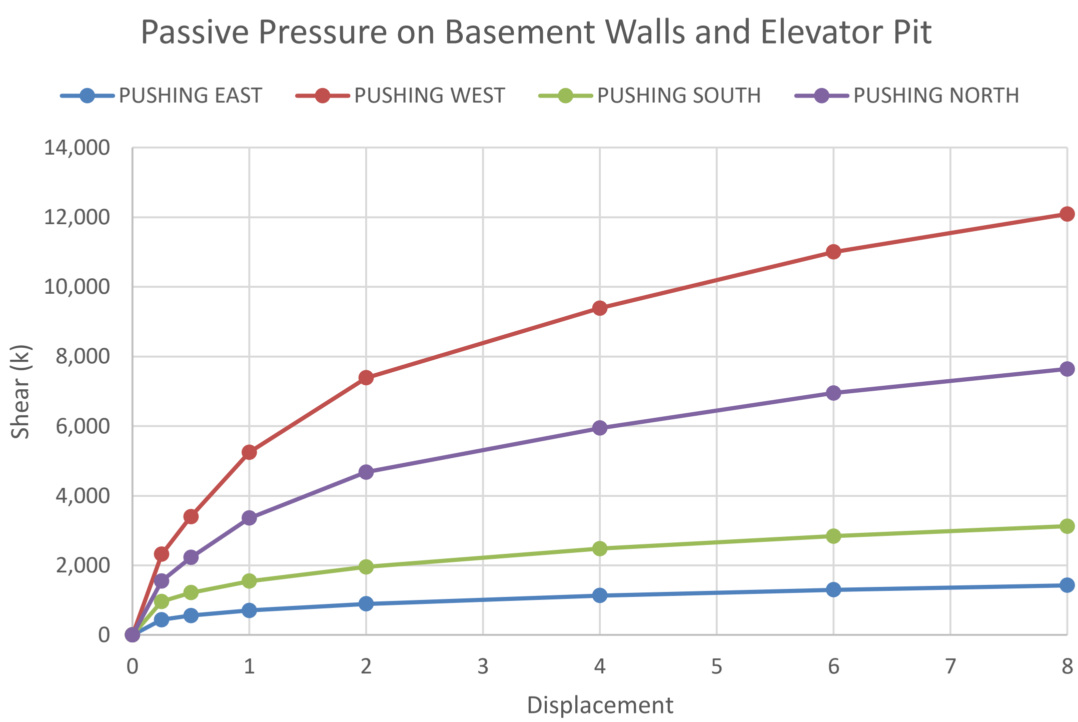


Figure 17. Passive Soil Resistance to Lateral Loads

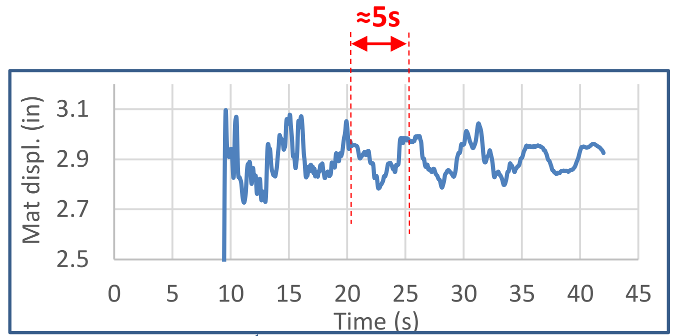
Bounding Earth Pressures

Existing Foundation:

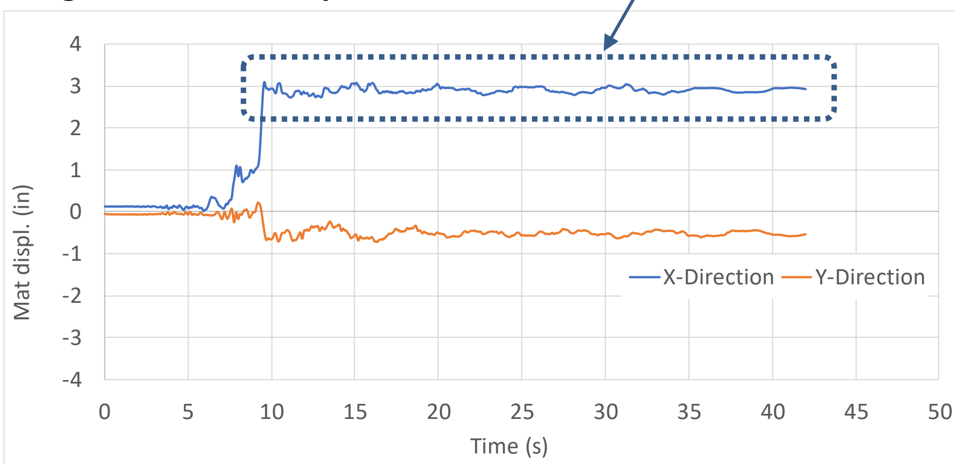
- Unbalanced Soil Pressures NOT Included
- Passive Soil Pressure NOT Included
- LERA Calculated Gravity Loads
- Initial Pile Head Rotations Included
- Lower Bound Soil Stiffness

Ground Motion	Average Displacement Across the Mat			
	Ux (in)		Uy (in)	
	Pushing East	Pushing West	Pushing North	Pushing South
RSN178_IMPVALL	2.09	-0.89	1.07	-0.97
RSN184_IMPVALL	NC	NC	NC	NC
RSN316_WESMORL	NC	NC	NC	NC
RSN802_LOMAP	NC	NC	NC	NC
RSN832_LANDERS	NC	NC	NC	NC
RSN1163_KOCAELI	2.70	-0.15	0.74	-3.51
RSN1261_CHICHI	4.71	-0.37	0.33	-1.13
RSN1511_CHICHI	3.02	-0.38	0.26	-2.58
RSN5827_SIERRA	3.63	-0.23	0.22	-3.71
RSN6890_DARFIELD	2.64	-0.20	0.36	-4.11
RSN6959_DARFIELD	1.62	-0.23	0.23	-1.63
Average of 10 Converged Ground Motions	***2.91***	***-0.35***	***0.46***	***-2.52***

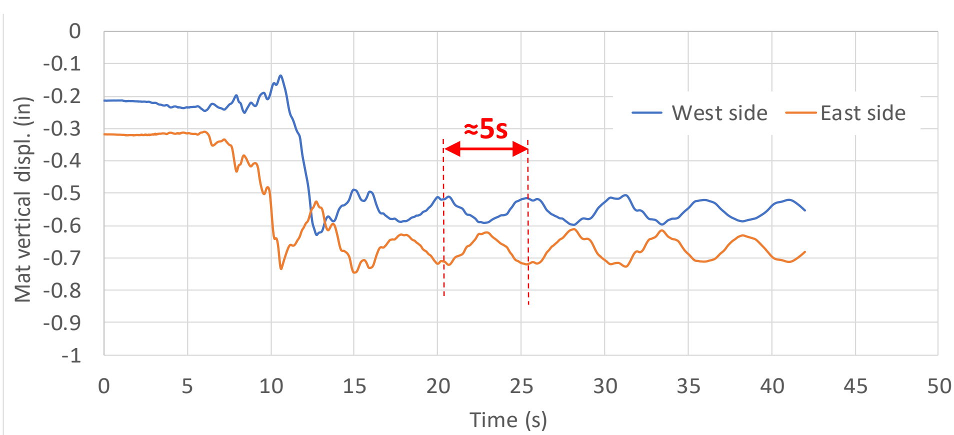
Figure 18. Nonlinear Time History Analysis Results Excluding Earth Pressures



Mat Average Horizontal Displacement:

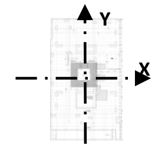


Mat Vertical Displacement:



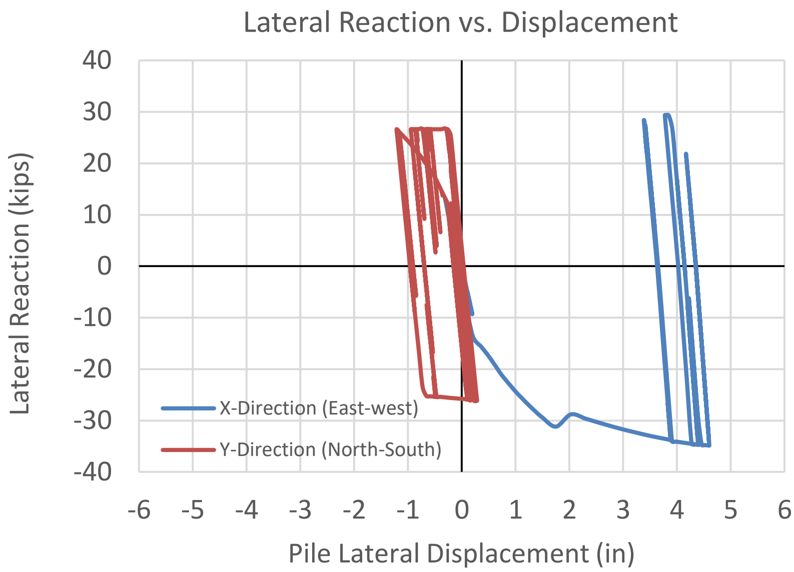
<u>RSN316 WESMORL – Representative of AVERAGE Motion</u>

Figure 19. BSE-1N (2/3*MCE) NLRHA Results – Mat Oscillations



Representative Hysteretic Response Plots

<u>RSN184 IMPVALL – Representative of STRONGEST Motion:</u>



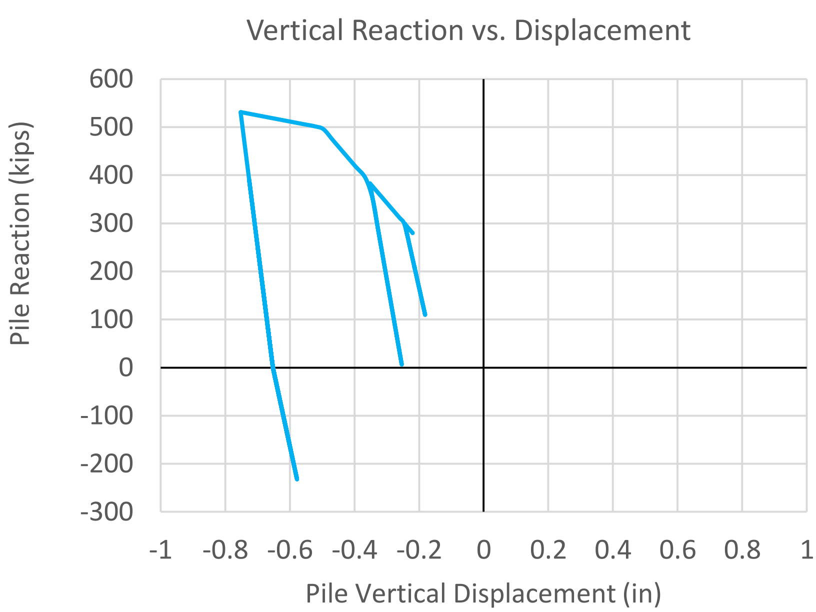
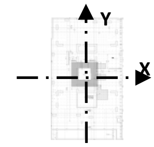


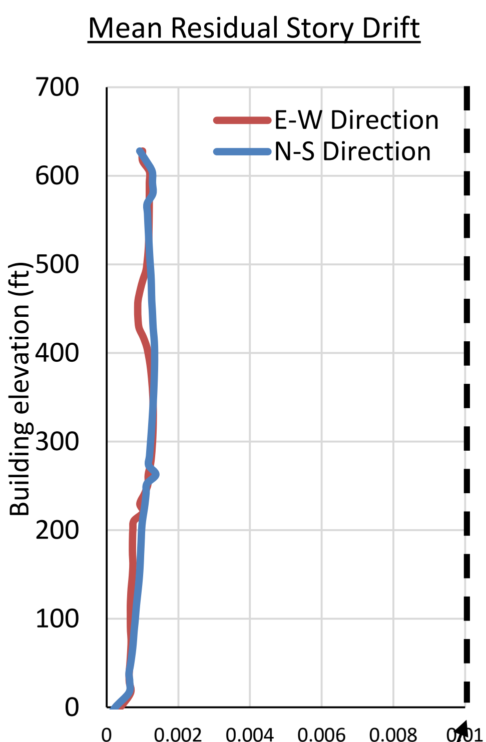
Figure 20. Representative Pile – Hysteretic Response

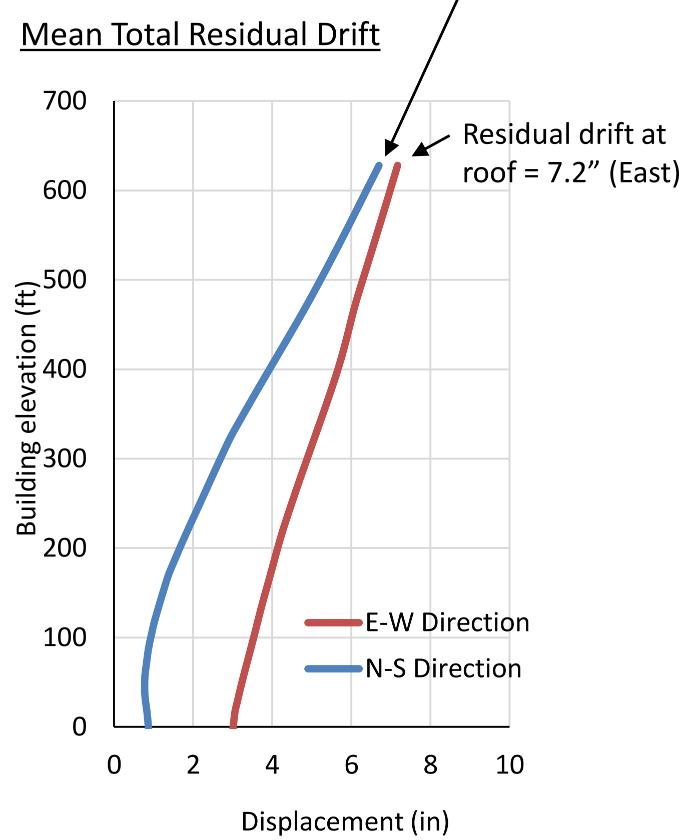


RESIDUAL BUILDING DRIFT

2/3 MCER (BSE-1N)

Residual drift at roof = 6.6" (primarily South)



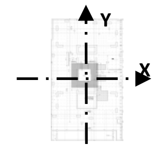


Acceptance Criteria per PEER v2.03:

6.7.3 Residual Story Drift

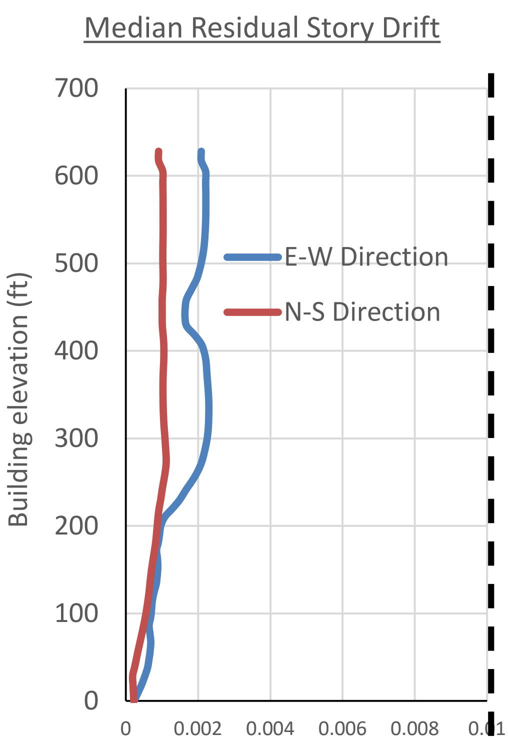
In each story, the mean of the absolute values of residual drift ratios from the suite of analyses shall not exceed 0.01.

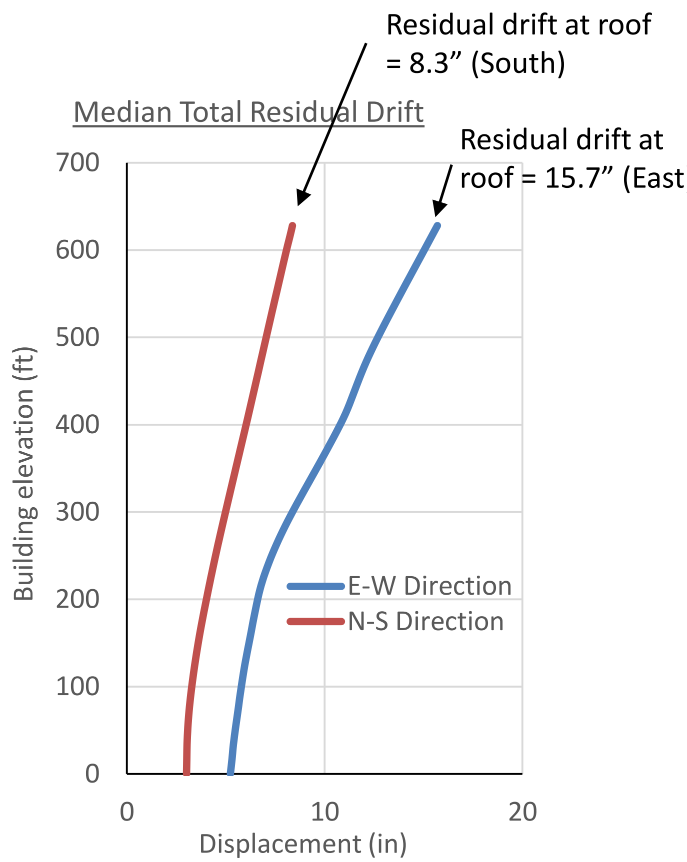
Figure 21. BSE-1N (2/3*MCE) NLRHA Results – Residual Building Drift



RESIDUAL BUILDING DRIFT

MCER (BSE-2N)





<u>Acceptance Criteria per PEER v2.03:</u>

6.7.3 Residual Story Drift

In each story, the mean of the absolute values of residual drift ratios from the suite of analyses shall not exceed 0.01.

Figure 22. BSE-2N (MCE) NLRHA Results – Residual Building Drift