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31 July 2018

Ms. Naomi M. Kelly
City Administrator
Office of the City Administrator
1 Dr. Carlton B. Goodlett Place, City Hall, Room 362
San Francisco, CA 94102

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 Information from the Millennium Tower Homeowners Association

Dear Ms. Kelly and Mr. Hui:

As requested in your June 5, 2018 letter to Mr. Vision Winter, we are providing you with 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.

The evaluation accomplished is in accordance with the procedures outlined in the California Existing Building Code and ASCE 41 for performing a seismic evaluation of an existing building. These procedures, as noted in Figure 1, include a review of building performance for two seismic hazard levels – your requested MCEr shaking, evaluated against Collapse Prevention acceptance criteria, and a less severe, more frequent event two-thirds as strong as the MCEr shaking, evaluated against the more stringent Life Safety acceptance criteria. Our review accounted for the impacts from the settlement and tilting of the tower that has transpired to date (see Figure 2) as well as potential future settlement and tilting that may occur over the next 3 years. This is a timeframe consistent with the estimated duration of the retrofit construction. Also, consistent with ASCE 41 requirements, our review accounts for soil variability and uncertainty using an upper- and lower-bound approach to defining stiffness and capacity for the existing tower pile foundation.

Our analysis of the Millennium Tower has been focused on designing a retrofit and evaluating the performance of the retrofit. We have not, therefore, prepared a report of existing conditions. Regardless, we have endeavored to address your questions and requests for supporting documentation below.

1. **Provide a description of the nonlinear analysis models used to evaluate the building, including descriptions of the software used and the types of components employed**

   In developing our retrofit design, we used a suite of analysis programs, including SAP2000, ETABS, and PERFORM 3D. For the evaluation of the existing building, we used our ETABS model to complete the requested non-linear time history seismic analysis. The ETABS model includes nonlinear representation of the strength, stiffness, and ductility of the various superstructure framing elements using properties based on data provided in ASCE 41 and a selection of commonly referenced research articles for nonlinear analysis. Details on the superstructure modeling are provided in Figures 3-14.

   The dynamic vertical and lateral nonlinear strength and stiffness properties for the existing precast concrete piles were developed with ENGEO considering soil-structure interaction. ENGEO's soil strength and stiffness properties were based on detailed study of the underlying soils and they were able to provide a tighter bounding than what is required in ASCE 41 (see Figure 15). A further discussion of the lateral stiffness is provided in our response to number 3 below.

   We subjected the building model to a suite of 11 ground motions consistent with the current practice of design for tall buildings. These ground motions were selected by ENGEO and scaled using the site specific partially non-ergotic spectra they developed in accordance with ASCE 7-10 requirements. More details on the ground motions are provided in Figures 16-18.

Unbalanced active, seismic and passive soil pressures against the north and west basement walls, and passive pressure from the soil against the depressed section of mat around the elevator pits have been accounted for.

2. Plots of overall lateral building drifts and story drift ratios under MCEr shaking

See Figure 19 for our plots of the average superstructure story drift under MCEr shaking for the ground motions that converged. We found that, for ground motions that converged, the expected building drift meets the current accepted drift limits for new buildings. Further discussion about the MCEr analysis results is included in the following sections.

3. Sample lateral capacity curves for existing piles (pile head shear versus displacement)

The existing pile lateral performance is affected by the soil properties, the amount of settlement induced rotation experienced at the top of the pile, the vertical load in the pile, the as-built pile construction record, and the structural non-linear load deformation characteristics of the pile. These variables have all been accounted for in our analysis. The procedure for developing the lateral capacity curves is provided in Figure 20 and sample pile lateral capacity curves for varying axial loads and assumed soil properties are provided in Figures 21 and 22.

The detrimental effects of the uneven settlement of the tower mat and resulting rotation at the top of the pile head are significant. We observed that even modest pile head rotations have an effect on pile lateral capacity. A pile head rotation of just 0.25 degrees materially reduces the lateral load carrying capacity of a pile. A review of the impact of the uneven settlement and the corresponding calculated pile head rotations in each principal direction is included in Figure 23. Taken collectively, the lateral load capacity of the pile group has been materially reduced in the east direction and even more so in the south direction due to the effects of the ongoing settlement and tilt. Additional uneven settlement will continue to degrade the capacity of the piles.

4. Provide building pushover curves for each principal direction (NS/EW) indicating maximum displacement at top of piles for each record in suite of MCE ground motions and the average of suite and identifying when (1) the first pile yields where it connects to the mat, (2) the first pile hinge experiences significant (>50%) strength degradation, and (3) two hinges form in a critical pile (precursor to a pile side-sway mechanism). Report how many piles experience yielding at the mat connection, significant strength degradation, or double hinging under the average and maximum suite of MCE ground motions.

Global building pushover curves for each principal direction using lower bound soil properties, pile loading based on our calculated gravity distribution, and accounting for passive pressure have been provided in Figures 25-26 & 29-30. We have indicated your requested points (1) through (3) on each curve as well as the maximum displacement at the top of the piles for each ground motion record. Tables with these displacements and the corresponding maximum base shears for each ground motion have been provided in Figures 24 and 28.

For the average and maximum BSE-1N (2/3*MCEr) event ground motions, we found that all the piles will experience yielding but none of the piles are expected to experience significant strength degradation or double hinging. Residual deformations in the foundation due to this yielding have been provided for a representative average ground motion in Figure 27. We find that the residual deformations are within a range that is consistent with Life Safety performance criteria.

For the BSE-2N (MCEr) event, based on our analysis to date, we found that 5 of the 11 ground motions failed to converge assuming lower bound soil properties, 4 of the 11 ground motions failed to converge assuming average soil properties, and 3 out of 11 ground motions failed to converge assuming upper bound soil properties. This performance in the east and south direction will continue to deteriorate with time as more non-uniform settlement occurs.

For the maximum ground motion, based on our current analysis, we found that all of the piles are expected to experience double hinging. Considering these non-convergences, we have chosen the median ground motion response for each direction (6th largest out of 11) to represent the average performance. For the median ground motion, based on our current analysis, we found that significant yielding is expected in all the piles and we estimate some of the piles will experience significant strength degradation and double hinging.

We also note that the use of a gravity loading distribution of forces in the piles for purposes of determining the foundations global backbone curve represents an optimistic view of the foundation capacity since it doesn't account for the amplification of force in the already degraded piles during eastward and southward seismic motions, nor does it account for the effect of pile tensions that will occur during seismic ground shaking. Evaluations of these effects are part of our ongoing efforts designing the retrofit.

5. Opinion of structural safety of the existing building

Based on these evaluations, our opinions about the performance and safety of the building are the following:

1. The tower and its foundations have experienced unacceptable settlements and tilting.

2. The tower structure from its mat foundation up has sufficient capacity to resist MCE ground shaking, and the settlement and tilting of the building has not affected this conclusion.

3. The non-uniform settlement and tilting of the building has introduced bending moments into the tops of the piles that diminish the capacity of the foundation. Subsequent non-uniform settlement and tilting will further diminish the capacity of the foundation.

4. The foundation in its current condition meets Life Safety criteria in the two-thirds MCEr shaking event, satisfying the reduced seismic performance objectives in the California Existing Building Code, and is suitable for occupancy now.

5. We believe a foundation retrofit is needed, and we recommend that such alterations be accomplished soon.

Very truly yours,

Daniel A. Sesil
LERA CONSULTING STRUCTURAL ENGINEERS, RLLP

DAS/jbs
Enclosure

cc: Mr. Vision Winter, O'Melveny & Myers

APPENDIX A: FIGURES AND PLOTS

Performance Objectives per ASCE 41-13:

[BS] TABLE 301.1.4.1
PERFORMANCE OBJECTIVES FOR USE IN ASCE 41 FOR COMPLIANCE WITH INTERNATIONAL BUILDING CODE-LEVEL SEISMIC FORCES

RISK CATEGORY (Based on IBC Table 1604.5)	STRUCTURAL PERFORMANCE LEVEL FOR USE WITH BSE-1N EARTHQUAKE HAZARD LEVEL	STRUCTURAL PERFORMANCE LEVEL FOR USE WITH BSE-2N EARTHQUAKE HAZARD LEVEL
I	Life Safety (S-3)	Collapse Prevention (S-5)
II	Life Safety (S-3)	Collapse Prevention (S-5)
III	Damage Control (S-2)	Limited Safety (S-4)
IV	Immediate Occupancy (S-1)	Life Safety (S-3)

[BS] TABLE 301.1.4.2
PERFORMANCE OBJECTIVES FOR USE IN ASCE 41 FOR COMPLIANCE WITH REDUCED INTERNATIONAL BUILDING CODE-LEVEL SEISMIC FORCES

RISK CATEGORY (Based on IBC Table 1604.5)	STRUCTURAL PERFORMANCE LEVEL FOR USE WITH BSE-1E EARTHQUAKE HAZARD LEVEL
I	Life Safety (S-3)
II	Life Safety (S-3)
III	Damage Control (S-2). See Note a
IV	Immediate Occupancy (S-1)

a. Tier 1 evaluation at the Damage Control performance level shall use the Tier 1 Life Safety checklists and Tier 1 Quick Check provisions midway between those specified for Life Safety and Immediate Occupancy performance.

Notes:

For the Millennium Tower site,

1. BSE-2N event is equivalent to the MCEr.
2. BSE-1N and BSE-1E event are equivalent to 2/3 of the MCEr event.

Foundation Acceptance Criteria per ASCE 41-13:

Table C2-4. Structural Performance Levels and Illustrative Damage

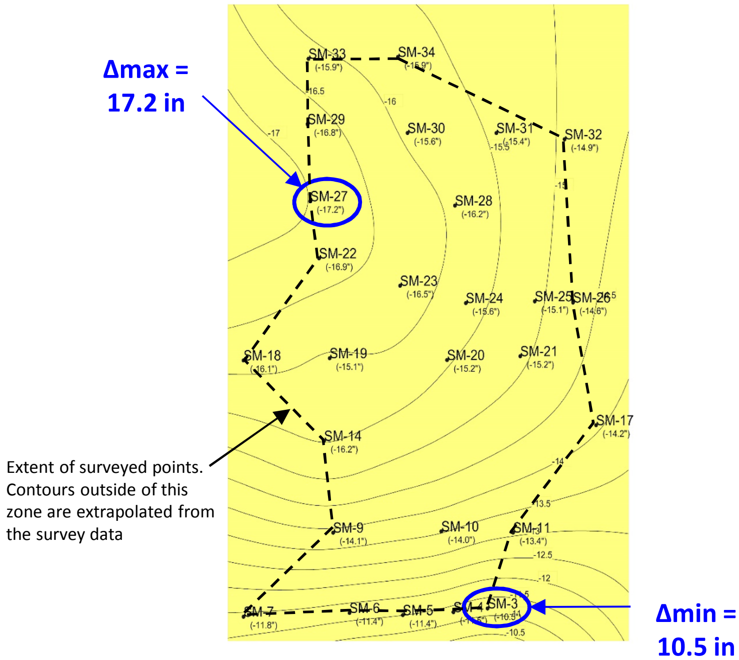
Seismic-Force-Resisting System	Type	Structural Performance Levels	
		Collapse Prevention (S-5)	Life Safety (S-3)
Foundations	General	Significant settlement and tilting of buildings with shallow foundations or buildings on liquefiable soils.	Localized settlement of buildings with shallow foundations.

Figure 1. 2016 San Francisco Existing Building Code Seismic Evaluation and Design Procedures (Section 301.1.4)

301 Mission Street
P0X021

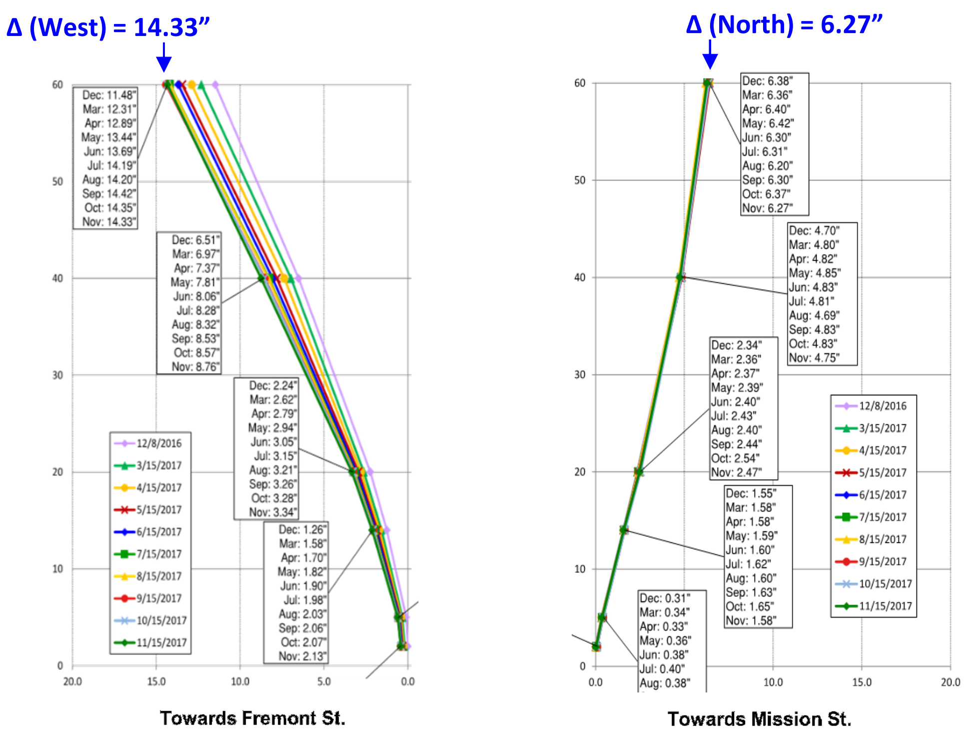
30 Jul 2018

TOTAL SETTLEMENT



Top of Mat Delta from Theoretical Elevation (in)
Langan Survey Data through 4/12/18

TOTAL TILT



East-West Direction (in) **North-South Direction (in)**

Offset from plumb - Langan Survey Data through 11/15/17

Figure 2. Millennium Tower Recent Settlement and Tilt Information

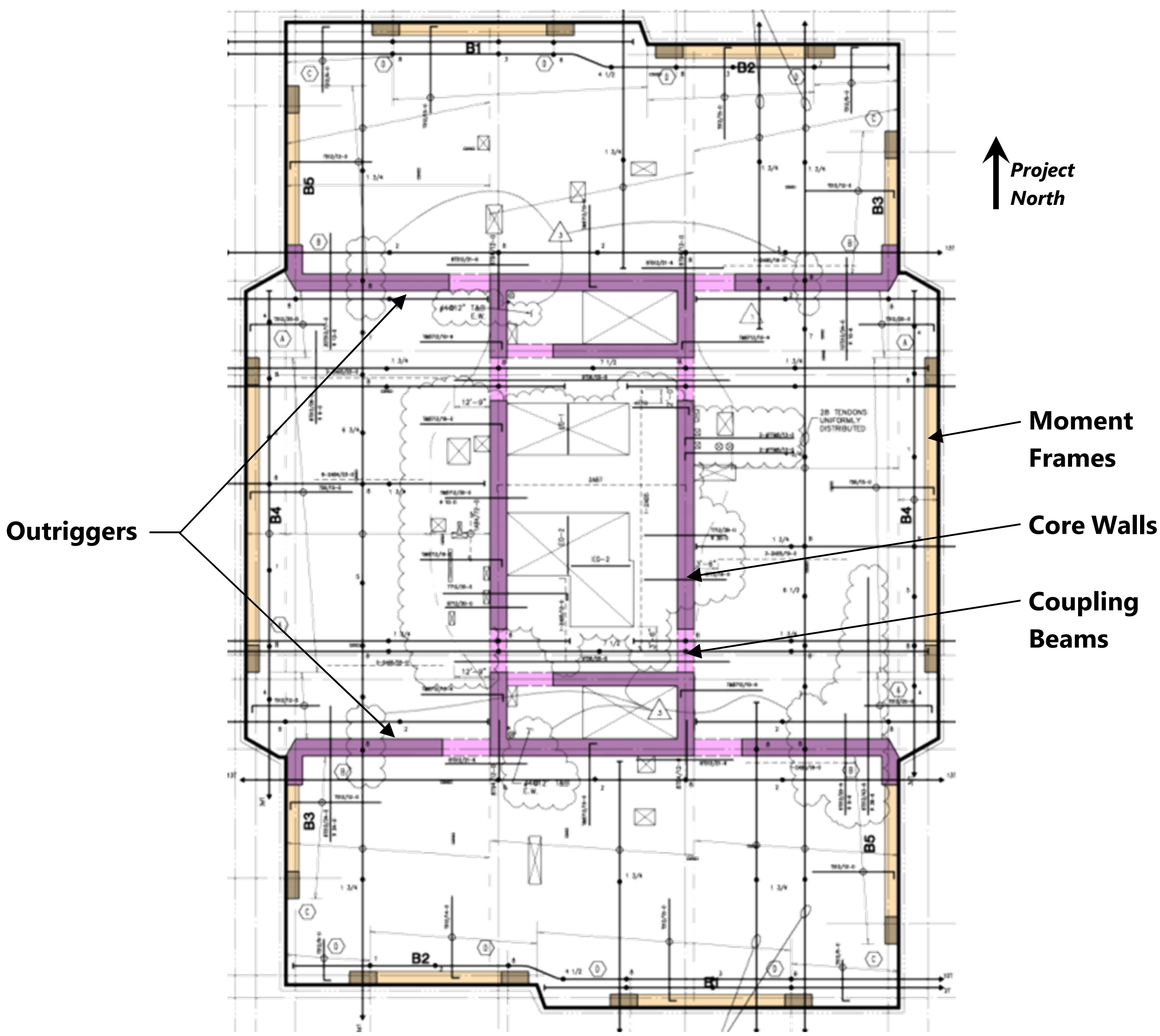


Figure 3. Typical Tower Floor Plan Highlighting Key Structural Elements (Source: Original Design Drawing S2-1.42.01)

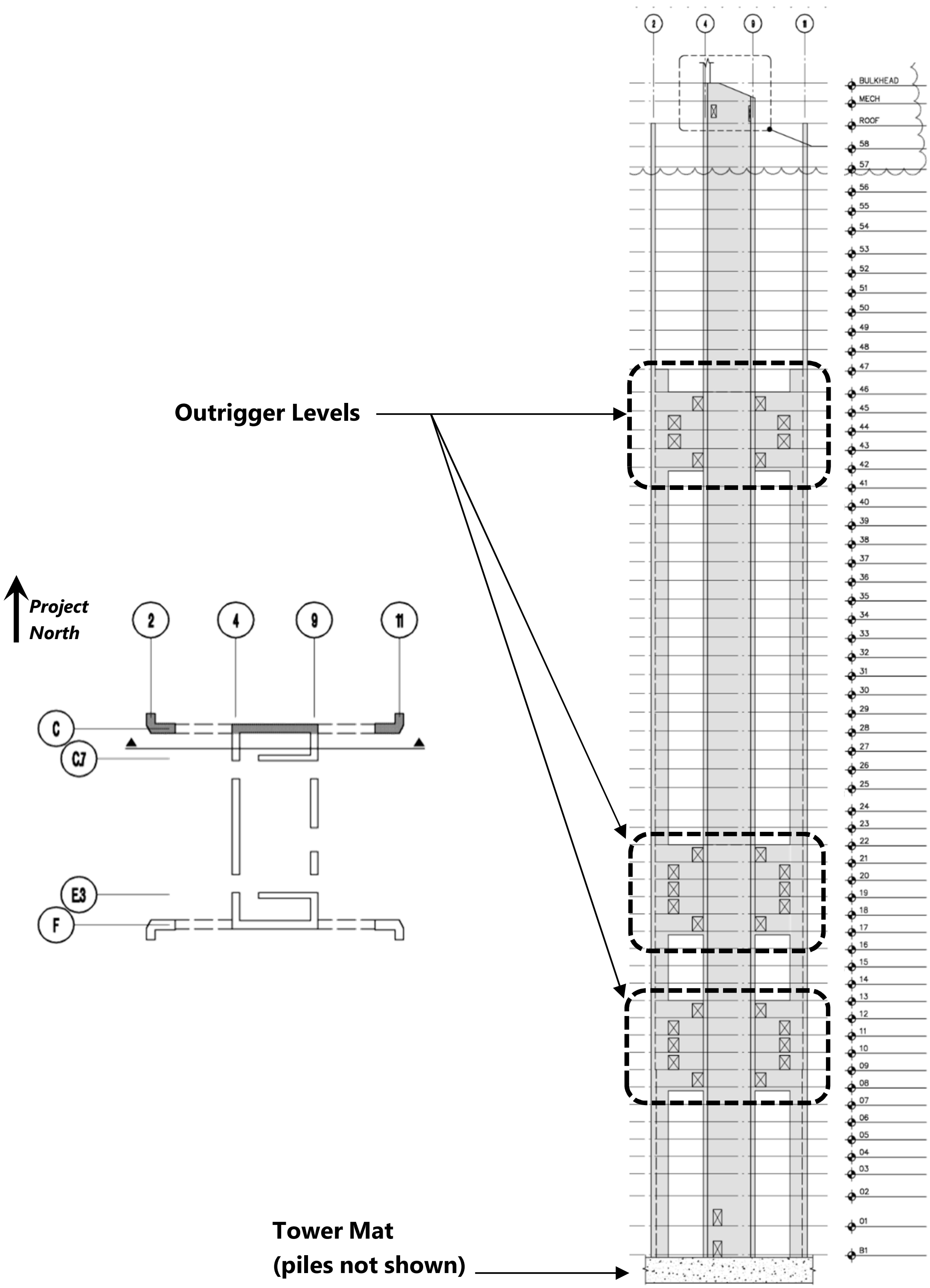


Figure 4. Tower Elevation at Core / Outrigger Line C (Source: Original Design Drawing S3-2.11)

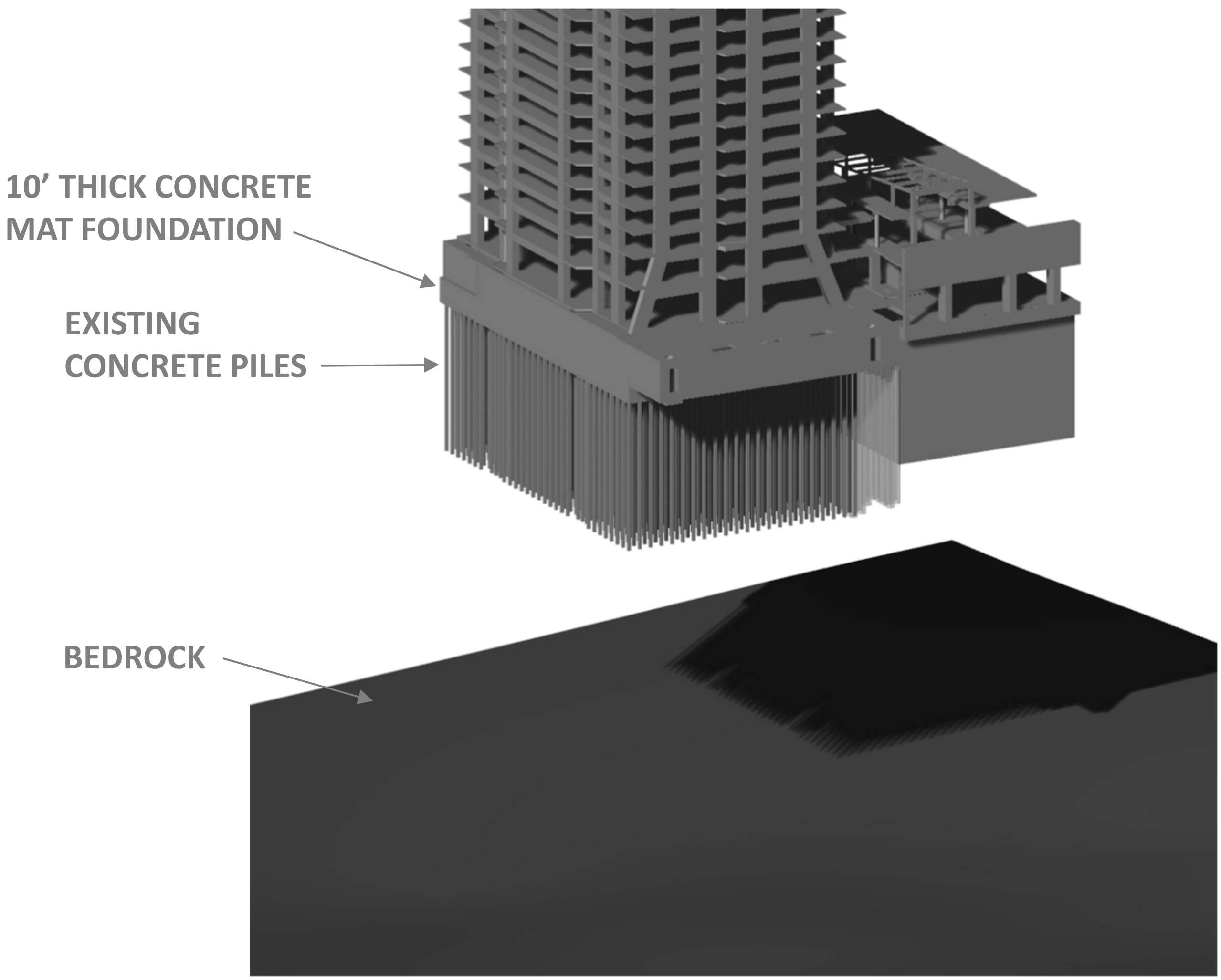


Figure 5. Millennium Tower Key Foundation Elements

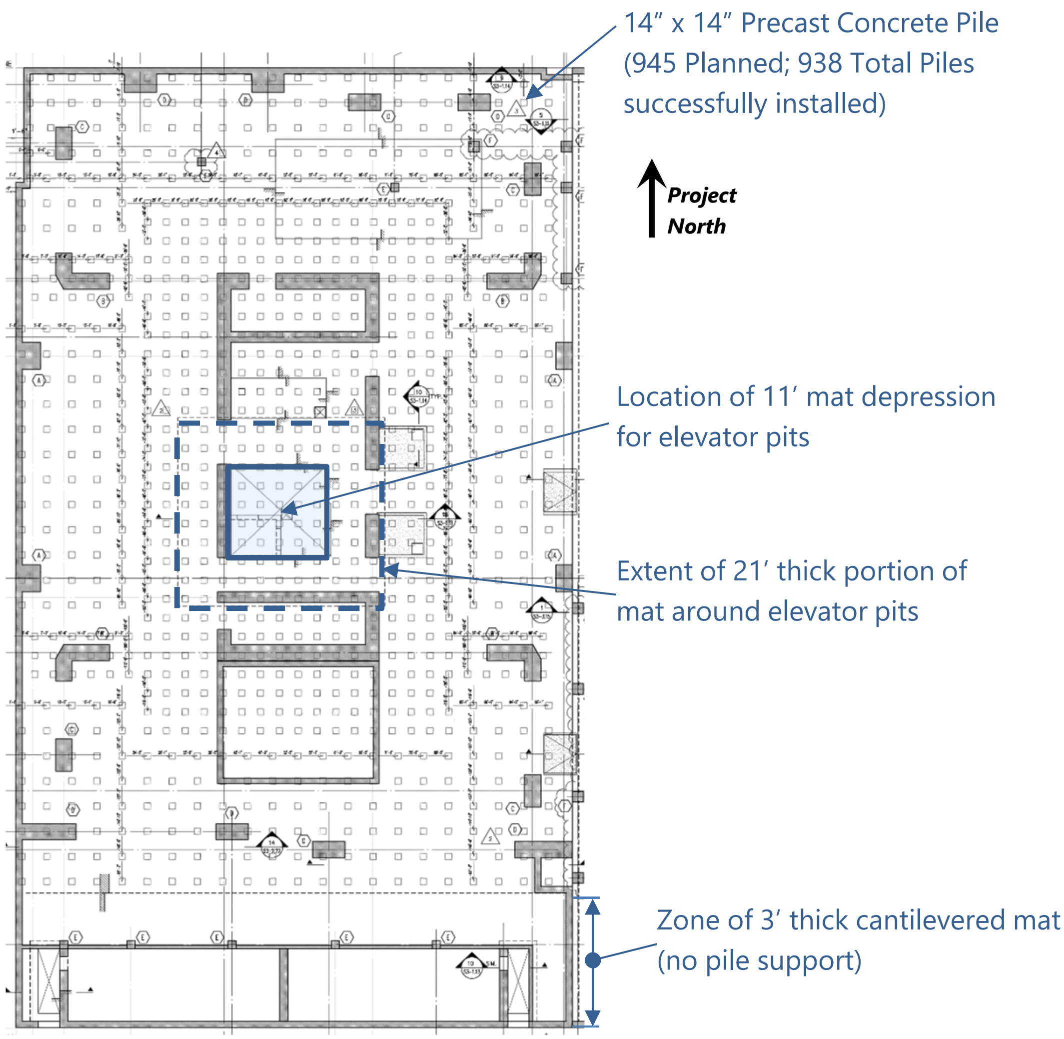


Figure 6. Pile Arrangement under Tower Mat
(Source: Original Design Drawing S2-0.B1.14)

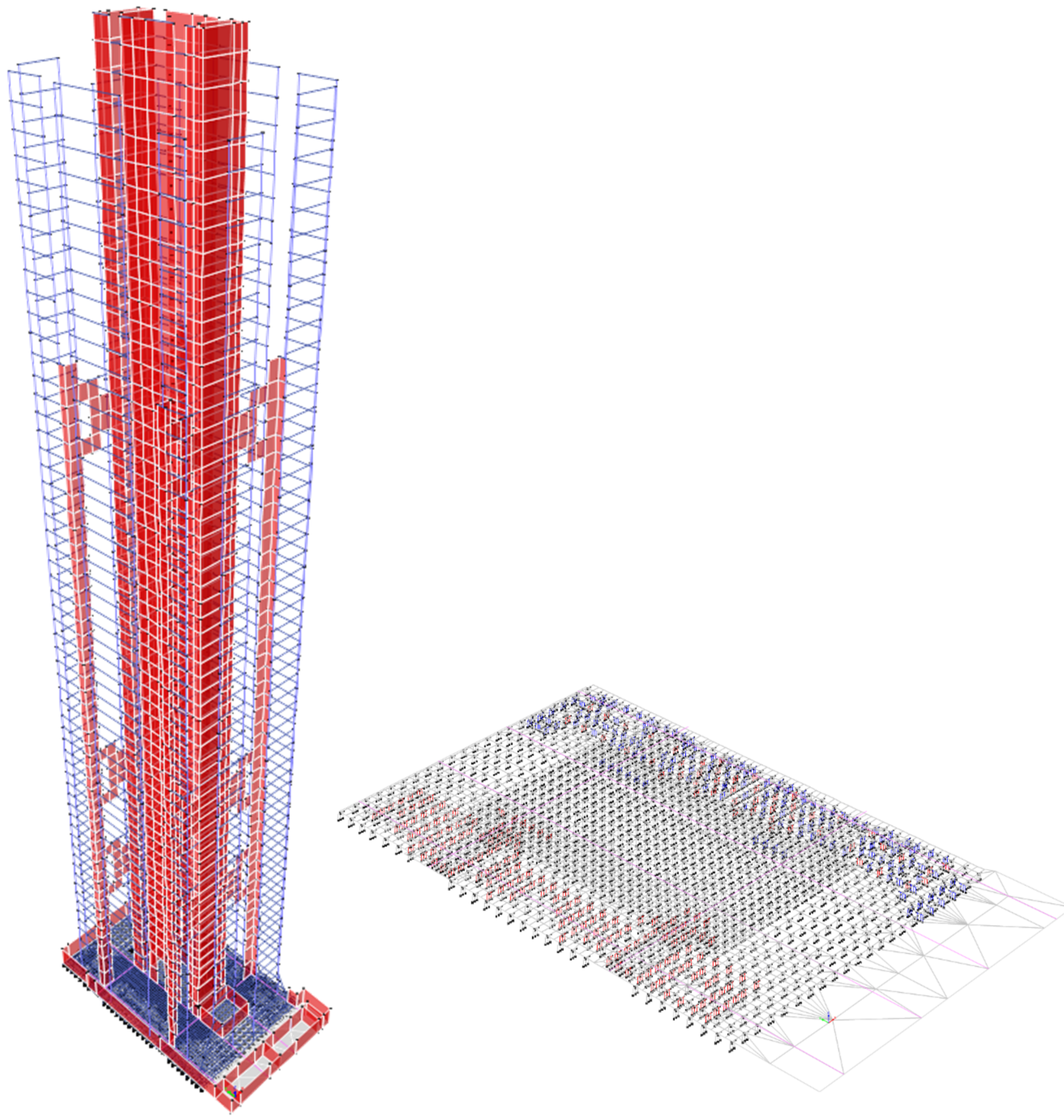
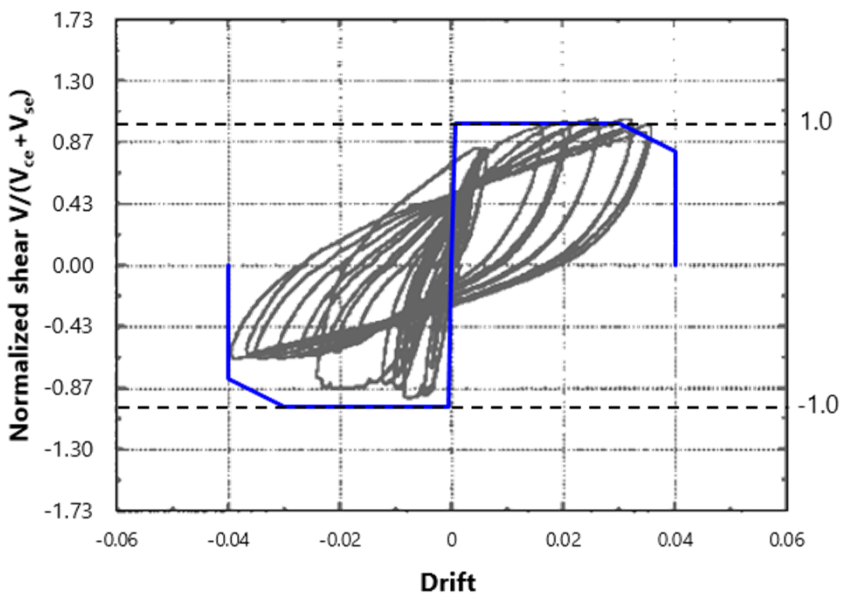


Figure 7. LERA ETABS Analysis Model

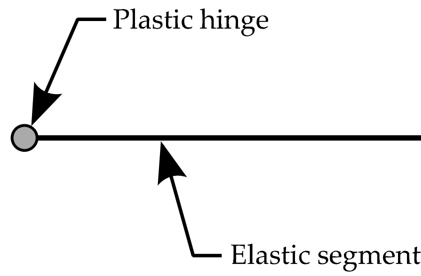
Inelastic shear panel with properties derived from research specimen with the closest aspect ratio (1:1)



Canbolat et al. (2005) Specimen 1

Canbolat, B.A., et al. (2005). "Experimental study on the seismic behavior of high-performance fiber reinforced cement composite coupling beams." *ACI Structural Journal*, ACI, 102(1), 159-166.

Figure 8. Modelling of Diagonally Reinforced Outrigger Walls



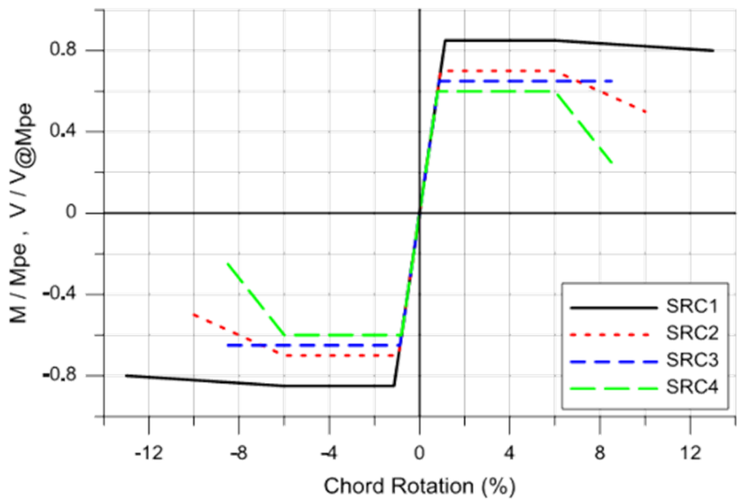


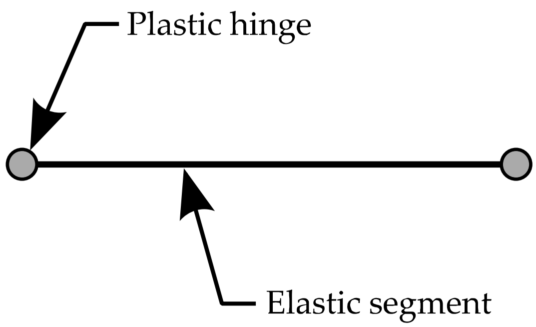
Table 7.5: Summary of Behavior Categories

Category		$A_s f_{ye} / C_b$	Wall Boundary Transverse Reinf.	Maximum Chord Rotation	% of $V_{ne,limit}$ used to Compute L_e	Modeling
I	A	≥ 1.0	SBE, OBE[1]	0.06	100%	SRC1
					80%	SRC2
	B		Other[2]	0.03	100%	SRC1
					80%	SRC2
II	A	≥ 0.5 & ≤ 1.0	SBE, OBE[1], Other[2]	0.06	100%	SRC3
					80%	SRC4
	B	≤ 0.5		0.03	100%	SRC3
					80%	SRC4
III		≤ 0.5	SBE, OBE[1], Other[2]	0.06	100%	SRC4

1: satisfies ACI 318-11 Section 21.9.6.5 with $\rho_{bound} > 400/f_y$
2: satisfies ACI 318-11 Section 21.9.6.5 with $\rho_{bound} \leq 400/f_y$

Motter, C.J., et al. (2017). "Steel-Reinforced Concrete Coupling Beams. II: Modeling." *J. Struct. Eng.*, ASCE, 143(3).

Figure 9. Modeling of Composite Coupling Beams



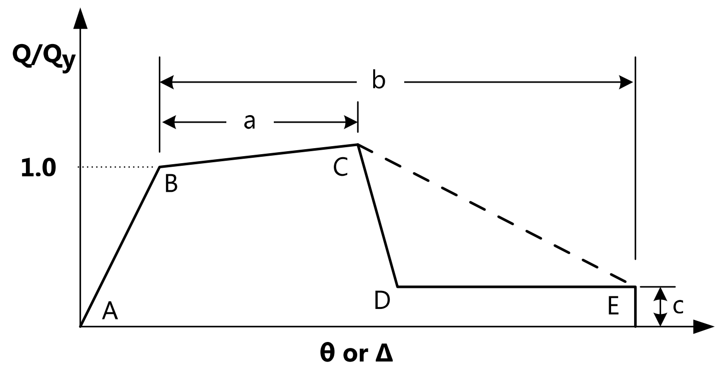


Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[a]: Plastic Rotations Angle (radians) *a*	Modeling Parameters[a]: Plastic Rotations Angle (radians) *b*	Residual Strength Ratio *c*	Acceptance Criteria[a]: Plastic Rotations Angle (radians), Performance Level IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d\sqrt{f'_c}}$ [d]						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ *d*/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > *d*/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ *d*/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > *d*/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]								
			0.015	0.03	0.2	0.01	0.02	0.03

NOTE: f'_c in lb/in.^{2} (MPa) units.
[a]Values between those listed in the table should be determined by linear interpolation.
[b]Where more than one of conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
[c]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ *d*/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.
[d]*V* is the design shear force from NSP or NDP.

ASCE 41-13 Moment-Rotation Relationship

Figure 10. Modeling of Conventional Coupling Beams and Moment Frame Beams

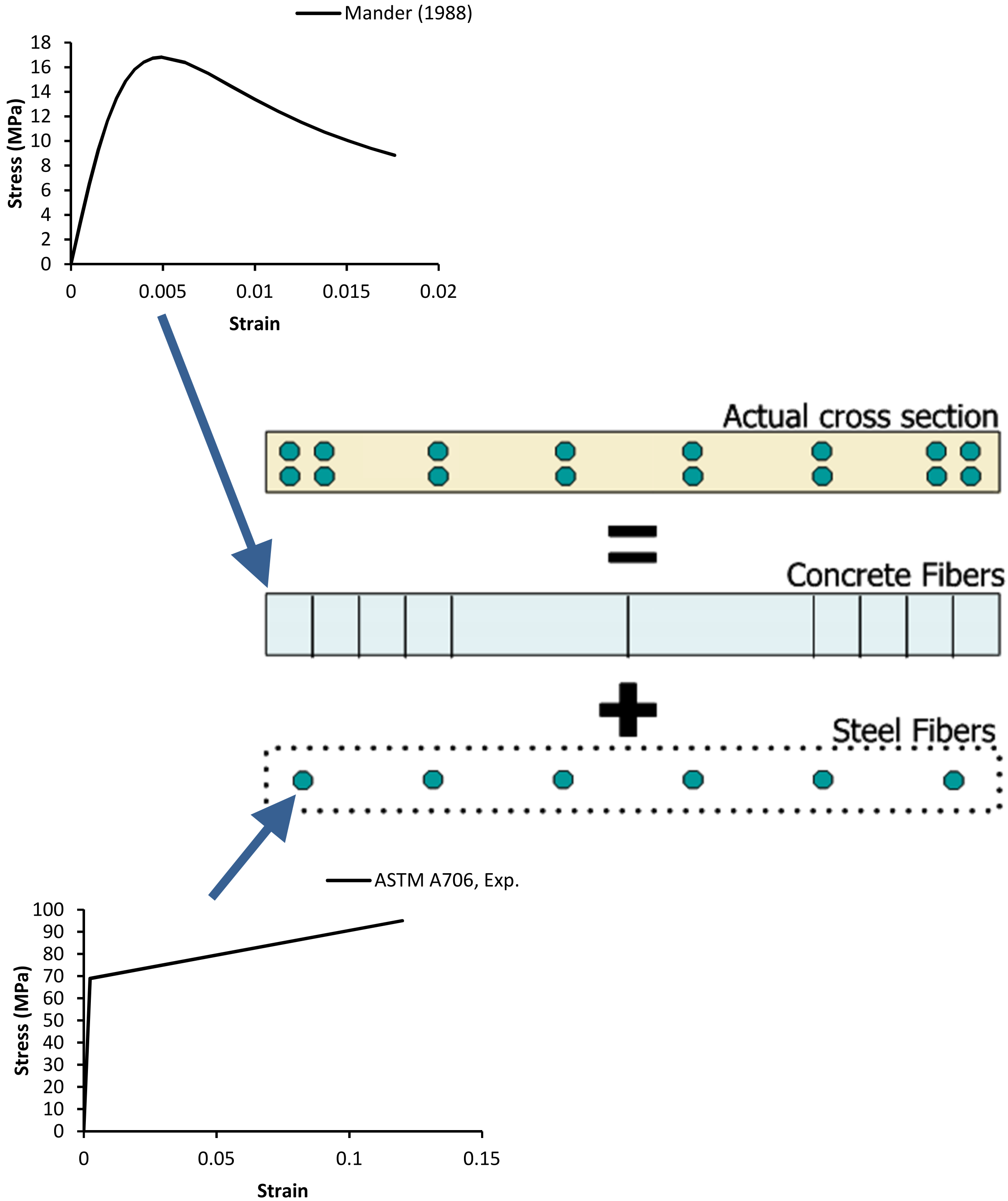
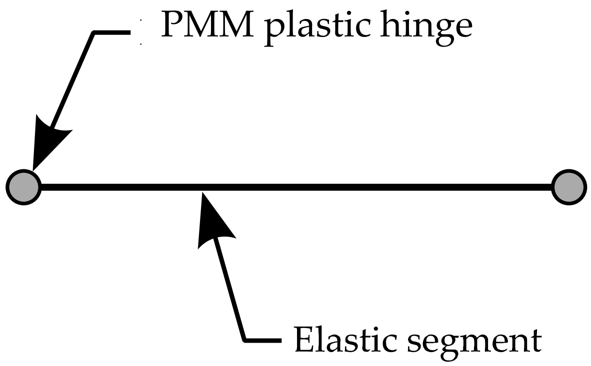
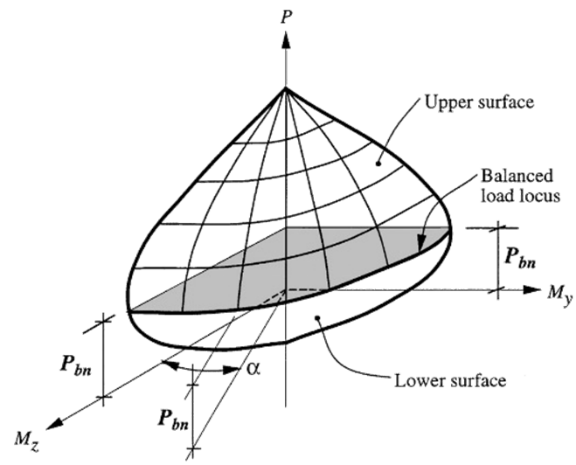
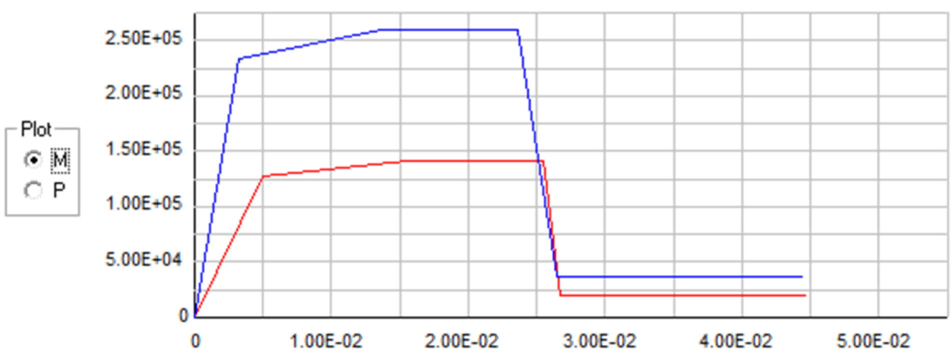


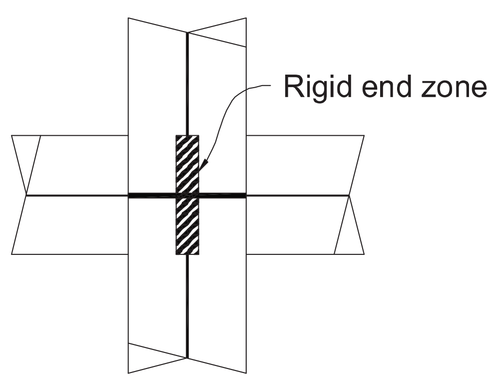
Figure 11. Modeling of Core Walls





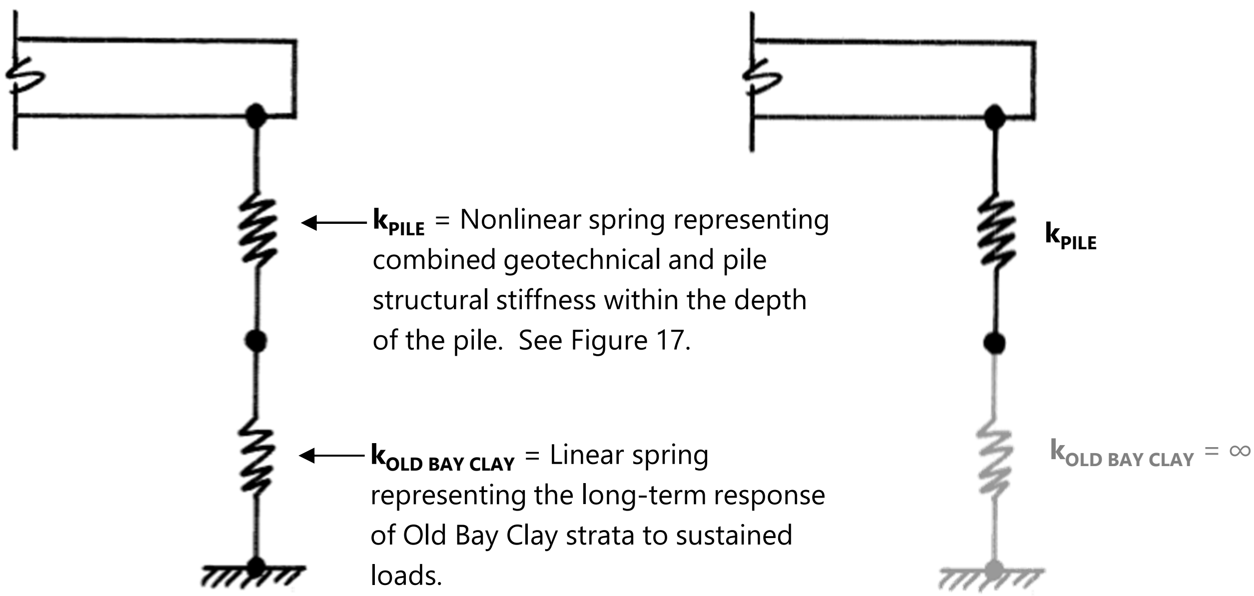


ASCE 41-13 Moment-Rotation Relationship



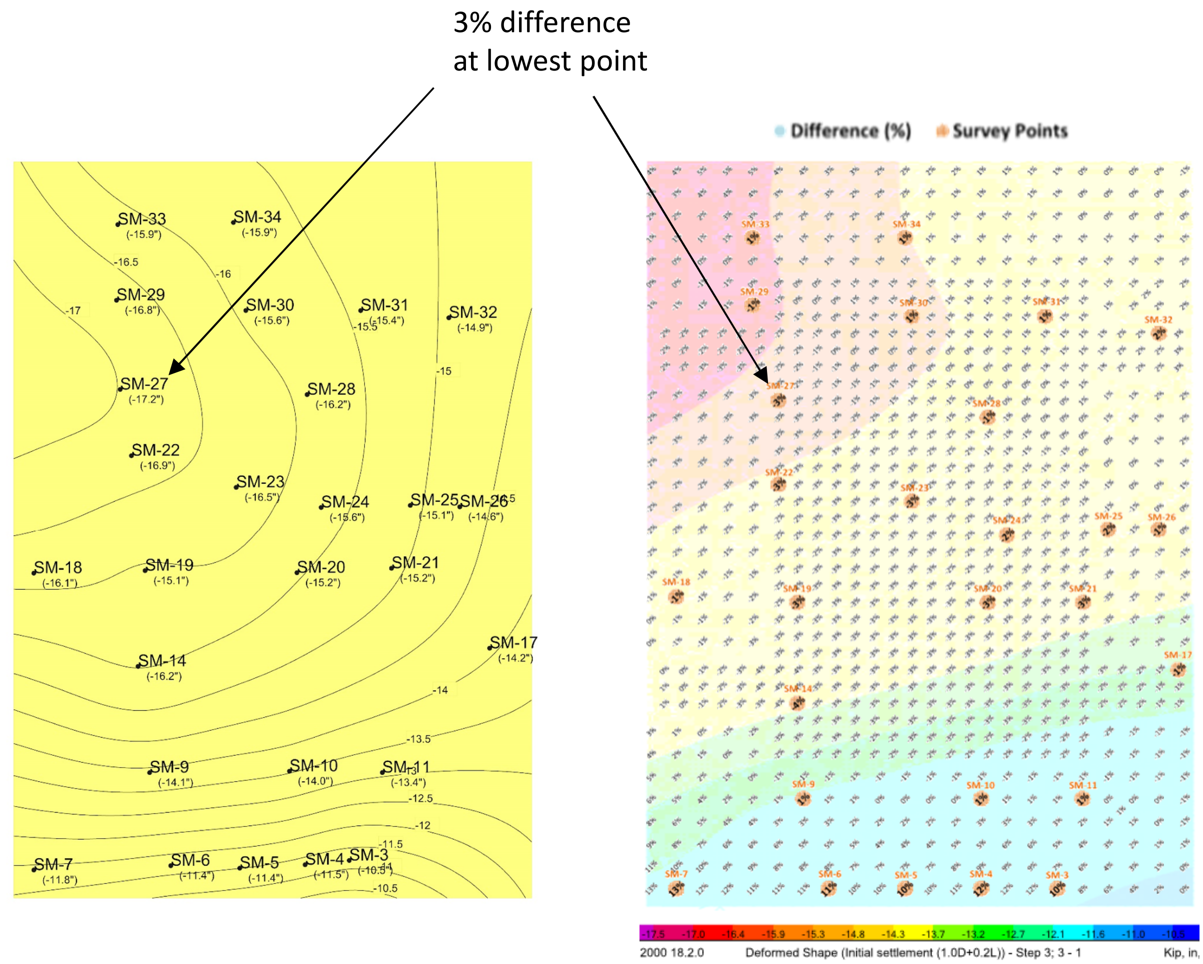
Recommended beam-column joint model per ASCE 41-13 when columns have been designed per strong column/weak beam provisions in the Code

Figure 12. Modeling of Moment Frame Columns



Static / Long Term Loads | **Dynamic / Short Term Loads**

Figure 13. Modeling of Existing Piles (Vertical Springs)



Total Surveyed Settlement (to April 12, 2018)

LERA ETABS Model – Gravity Loading

Figure 14. Difference between Surveyed Settlement and LERA Analysis

Upper and Lower Bound Pile Foundation Strength and Stiffness Requirements per ASCE 41-13 Figure 8-1(a):

ENGEO's analysis allowed for a tighter band of bounding than ASCE 41 (see Figure 22)

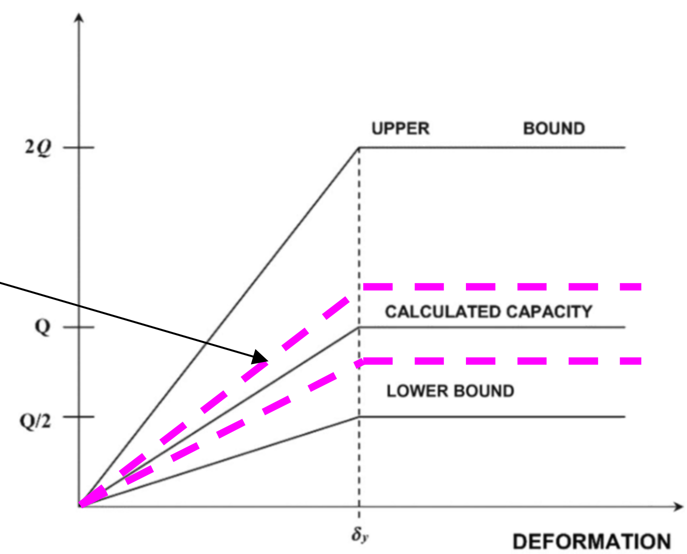
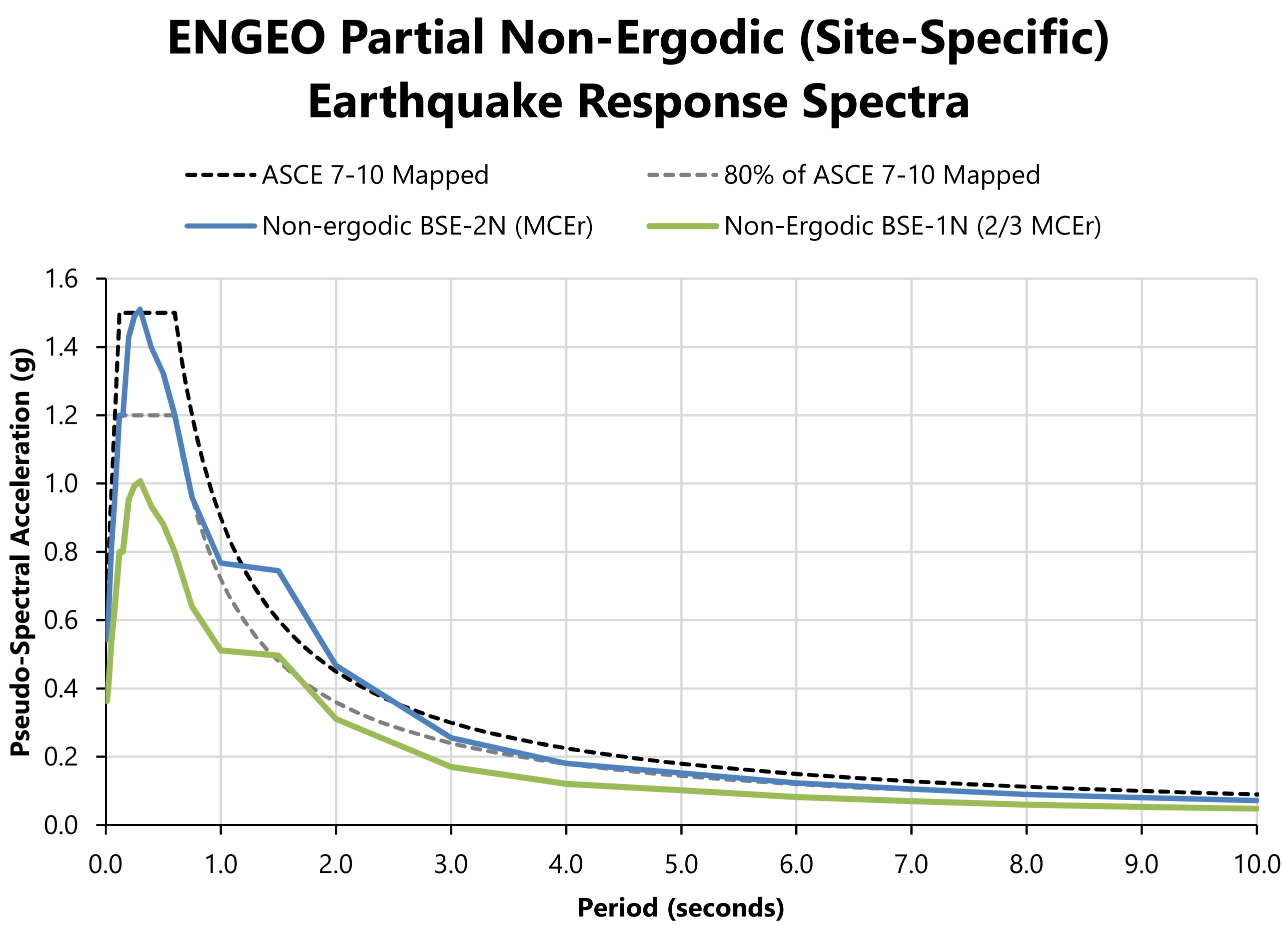


Figure 8-1(a)

Figure 15. Geotechnical Bounding per ASCE 41-13
Seismic Evaluation and Retrofit of Existing Buildings



<u>Building parameters for establishing spectra</u>

Parameter	Value
Risk category	II
Site class	D
Mapped MCE_R spectral acceleration at short periods	S_S = 1.50 g
Mapped MCE_R spectral acceleration at a period of 1 s	S_1 = 0.60 g
Fundamental Building Period [Note1]	T = 4.5s

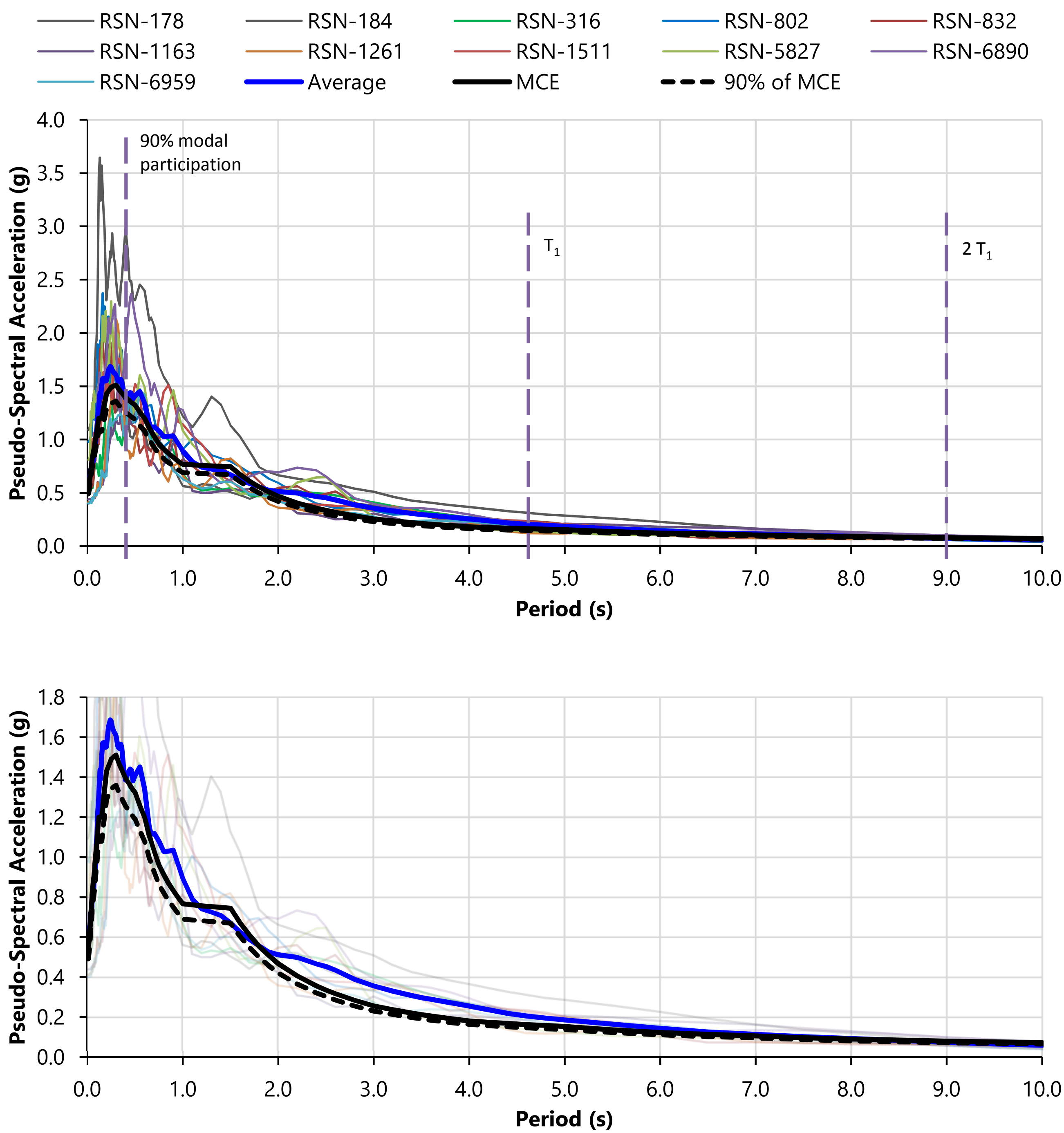
1. Building Period is established using expected material properties, component stiffness modification factors consistent with MCE level analysis (see Section 9.5), and excluding soil-structure interaction.

Figure 16. Non-Ergodic (Site-Specific) Earthquake Response Spectra

NO.	EARTHQUAKE	NGA #	PULSE PERIOD (sec)	MAG. (M_W)	R_{rup} (km)	FAULT TYPE	V_{S30} (m/sec)	D_{5-95} (sec)	BSE-2N (MCE) SF	BSE-1N (2/3*MCE) SF	Angle of Application (°)
1	"Imperial Valley-06"	178	4.501	6.53	12.85	strike slip	162.94	14.1	1.88	1.25	22/112
2	"Imperial Valley-06"	184	6.265	6.53	5.09	strike slip	202.26	7.00	2.20	1.47	90/180
3	"Westmorland"	316	4.389	5.9	16.66	strike slip	348.69	18.7	2.04	1.36	173/263
4	"Loma Prieta"	802	4.571	6.93	8.5	Reverse Oblique	380.89	9.4	1.88	1.25	0/90
5	"Landers"	832	-	7.28	69.21	strike slip	382.93	28.5	3.66	2.44	62/152
6	"Kocaeli_ Turkey"	1163	-	7.51	60.05	strike slip	354.37	36.7	3.83	2.55	106/196
7	"Chi-Chi_ Taiwan"	1261	-	7.62	56.06	Reverse Oblique	373.23	33.4	4.00	2.67	41/131
8	"Chi-Chi_ Taiwan"	1511	4.732	7.62	2.74	Reverse Oblique	614.98	29.5	1.50	1.00	136/226
9	"El Mayor-Cucapah_ Mexico"	5827	-	7.2	15.91	strike slip	242.05	34.5	1.50	1.00	46/136
10	"Darfield_ New Zealand"	6890	-	7.0	17.64	strike slip	204.00	20.0	2.50	1.67	92/182
11	"Darfield_ New Zealand"	6959	12.019	7.0	19.48	strike slip	141.00	30.5	1.21	0.81	90/180

Figure 17. Time History Ground Motion Records

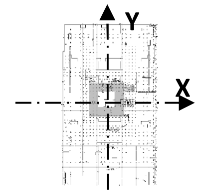
RotD100 (Maximum Direction) Spectra

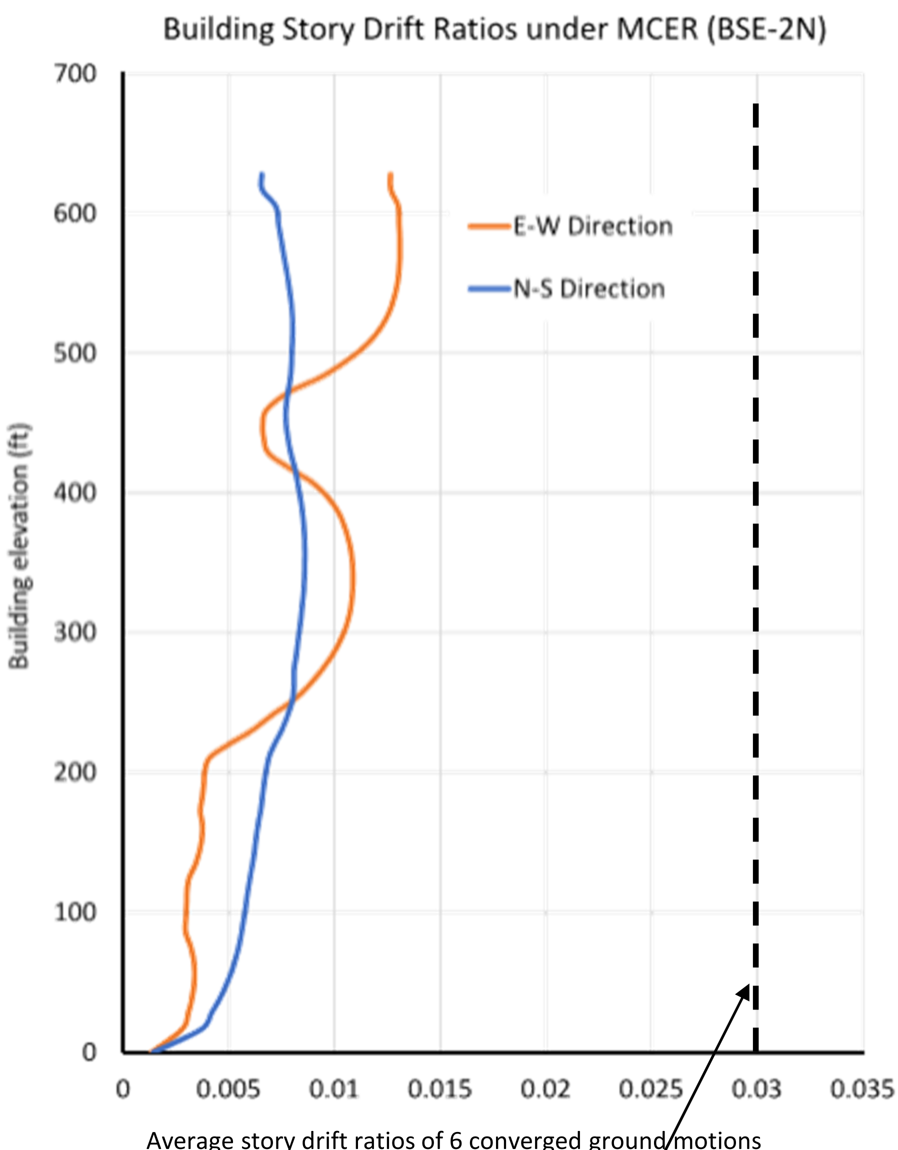


Ground motion records scaled such that the average RotD100 spectrum is not less than 90% of the target spectrum (non-ergodic MCE spectrum) within period range of interest per ASCE 7-16

Figure 18. Non-Ergodic Ground Motions

Pile Lateral Stiffness
Lower Bound





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

6.7.2 Peak Transient Story Drift

In each story, the mean of the absolute values of the peak transient story drift ratios from each suite or set of analyses shall not exceed 0.03.

Figure 19. BSE-2N (MCE_R) NLRHA Envelope Results – Story Drifts

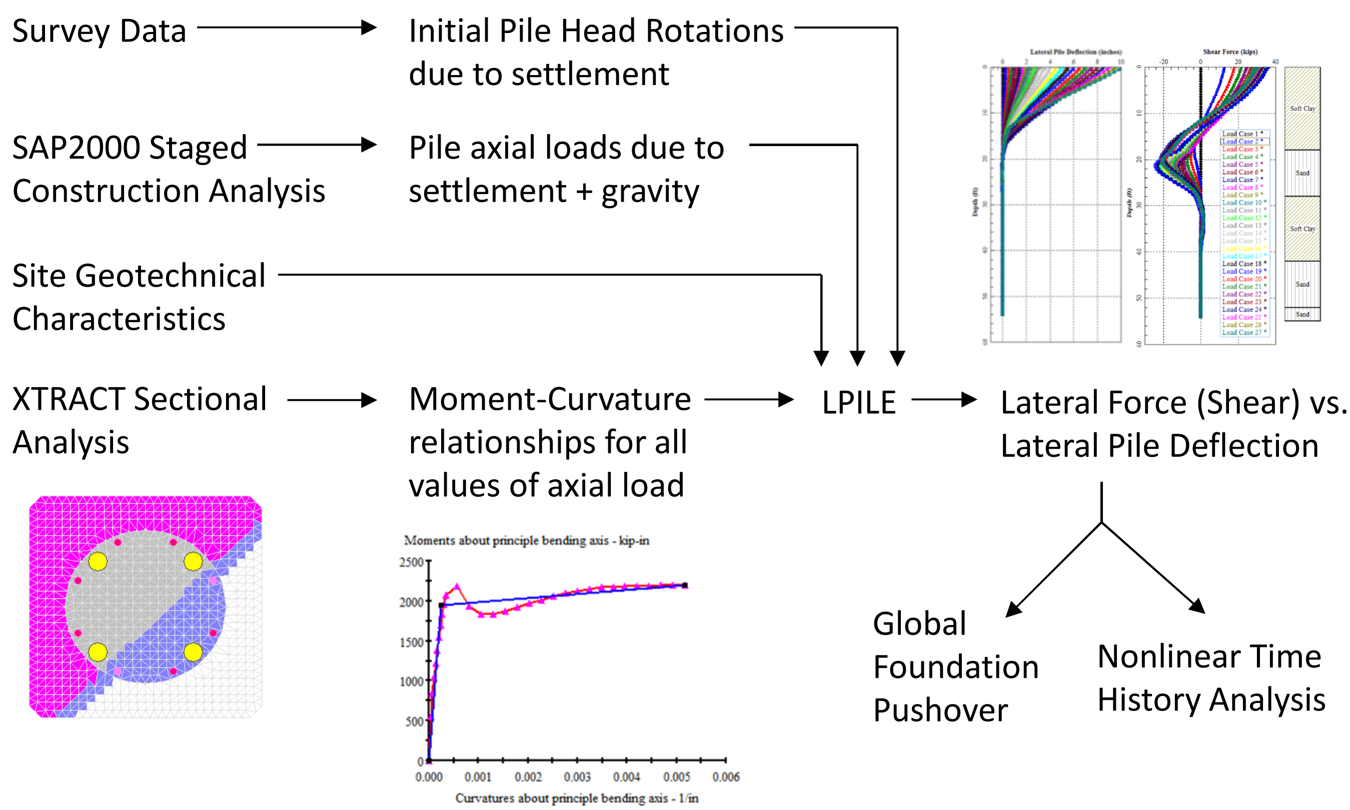


Figure 20. Laterally Loaded Pile Analysis Procedure

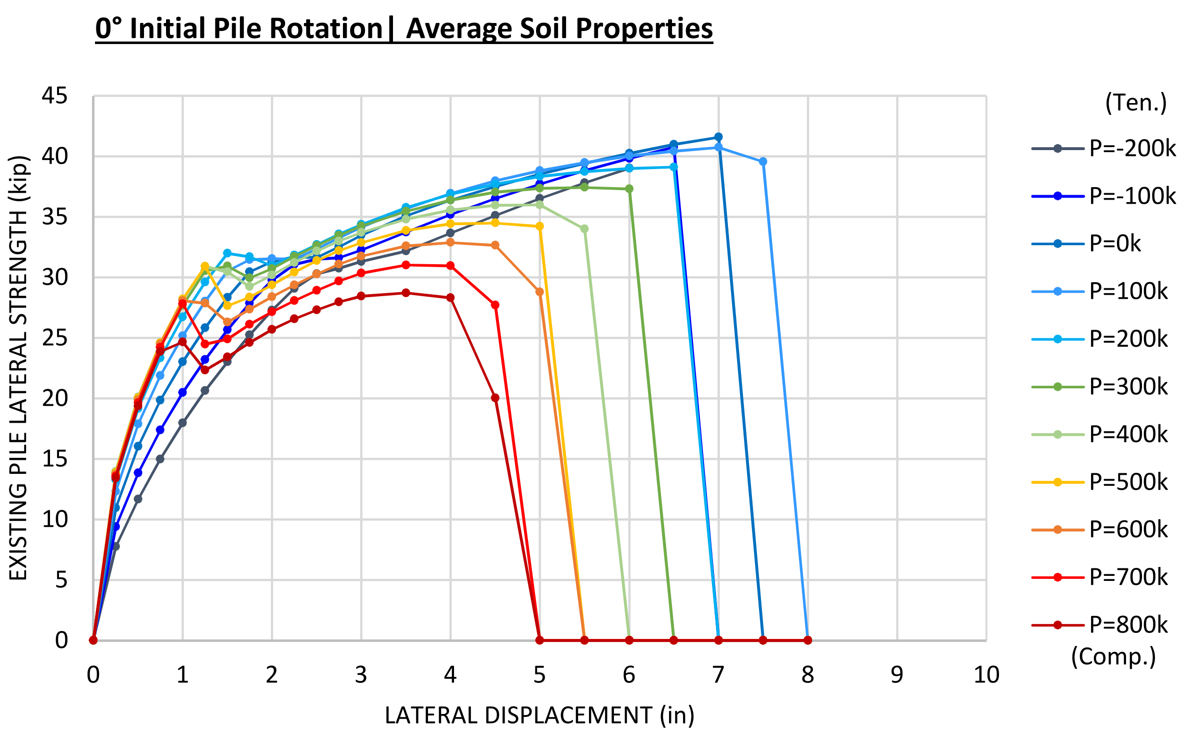


Figure 21. Individual Pile Lateral Stiffness / Capacity – Axial Load Variation

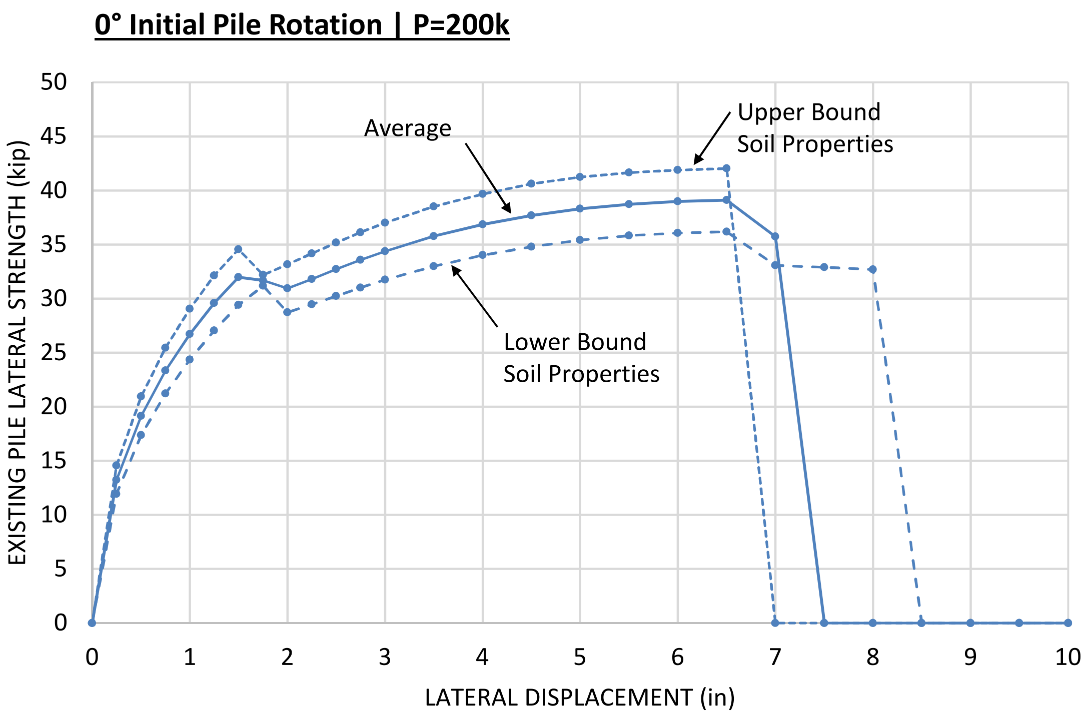


Figure 22. Individual Pile Lateral Stiffness / Capacity – Geotechnical Bounding

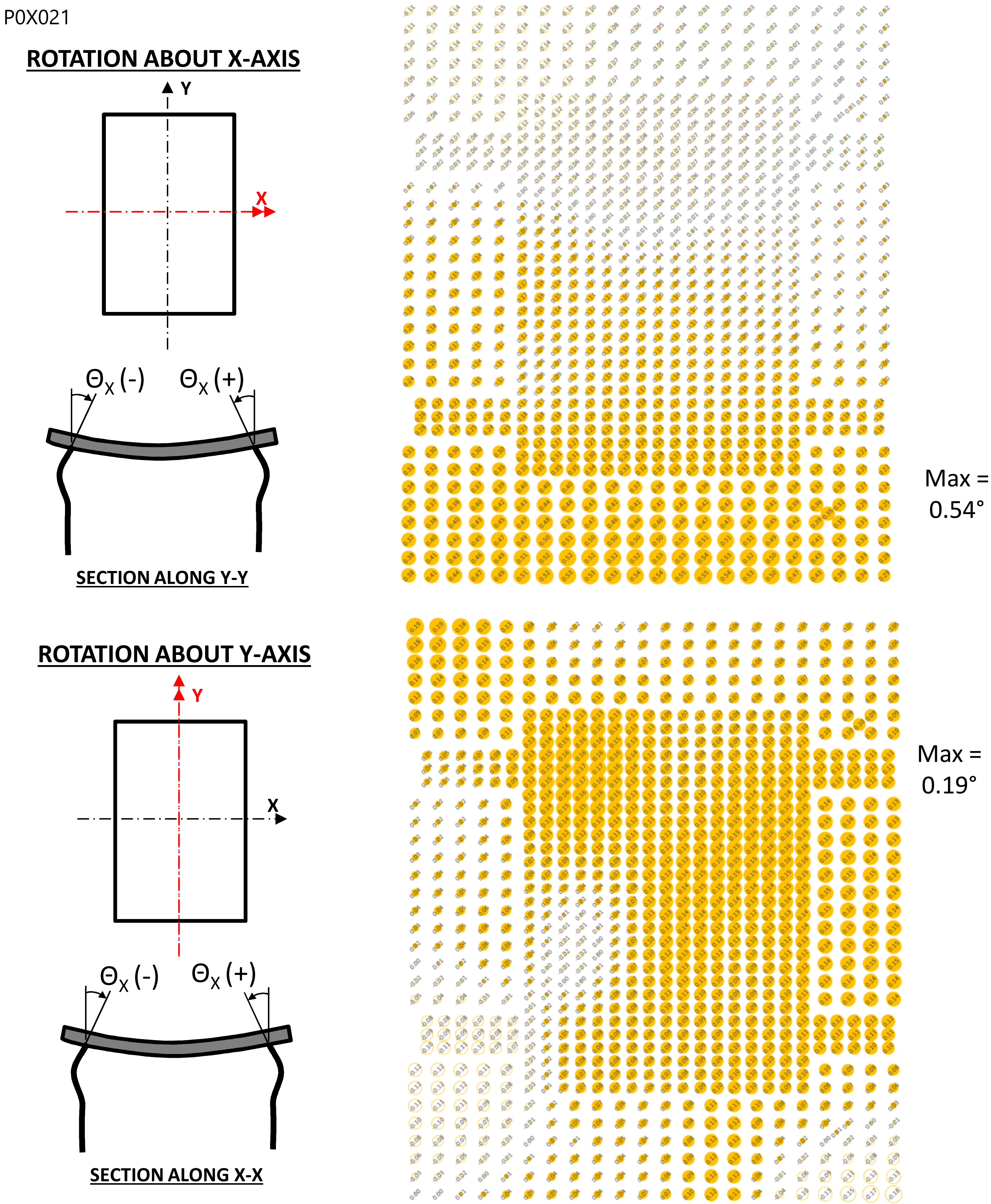
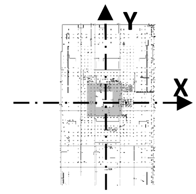


Figure 23. Initial Pile Head Rotations – (April 12, 2018)

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity



Base Shear:

Ground Motion	Fx (kip)		Fy (kip)	
	Pushing East	Pushing West	Pushing North	Pushing South
RSN178_IMPVALL	17,862	20,453	19,536	19,351
RSN184_IMPVALL	23,864	28,894	28,576	19,576
RSN316_WESMORL	21,589	17,019	15,935	15,851
RSN802_LOMAP	21,654	20,222	23,596	18,545
RSN832_LANDERS	24,017	26,924	20,814	20,886
RSN1163_KOCAELI	18,926	24,847	21,658	21,307
RSN1261_CHICHI	22,534	22,091	17,145	17,201
RSN1511_CHICHI	19,139	21,687	22,282	20,430
RSN5827_SIERRA	22,728	24,744	22,644	22,378
RSN6890_DARFIELD	18,053	20,383	28,147	22,193
RSN6959_DARFIELD	16,347	17,692	19,266	21,055
Average of 11 Ground Motions	**20,610**[1]	**22,269**	**21,782**	**19,889**[1]

Notes:
1) Base Shears include unbalanced active and seismic earth pressures

Displacements:

Ground Motion	Maximum Mat Displacement (Average of 4 Corners)				
	Ux (in)		Uy (in)		Uxy (in)
	Pushing East	Pushing West	Pushing North	Pushing South	Resultant
RSN178_IMPVALL	1.97	0.44	0.51	1.09	2.18
RSN184_IMPVALL	4.77	0.45	1.09	1.19	4.80
RSN316_WESMORL	3.12	0.10	0.28	0.74	3.17
RSN802_LOMAP	3.36	0.20	0.74	1.07	3.48
RSN832_LANDERS	4.64	0.23	0.29	1.43	4.81
RSN1163_KOCAELI	2.20	0.30	0.35	1.56	2.59
RSN1261_CHICHI	3.42	0.21	0.21	0.85	3.47
RSN1511_CHICHI	2.26	0.24	0.17	1.32	2.55
RSN5827_SIERRA	3.61	0.16	0.14	1.92	3.96
RSN6890_DARFIELD	1.92	0.23	0.50	1.68	2.08
RSN6959_DARFIELD	1.14	0.21	0.27	1.43	1.73
Average of 11 Ground Motions	***2.95***	***0.25***	***0.41***	***1.30***	***3.17***

Figure 24. BSE-1N (2/3*MCE) NLRHA Envelope Results – Base Shear and Displacements

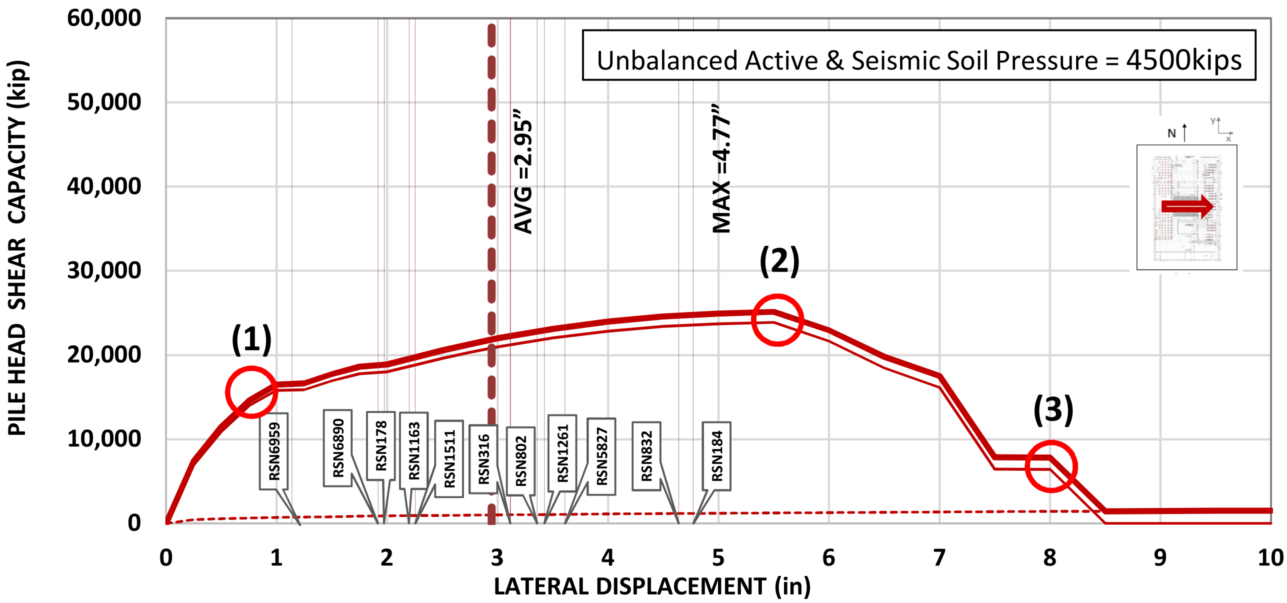
301 Mission Street

P0X021

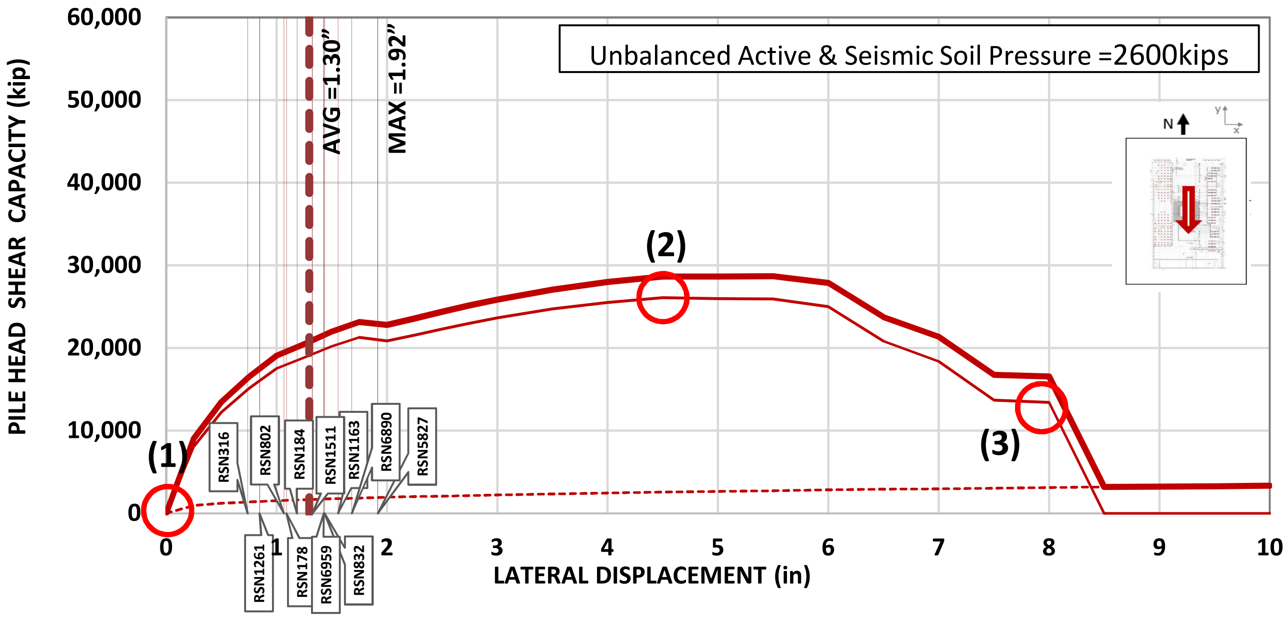
Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

30 Jul 2018

PUSHING TO EAST



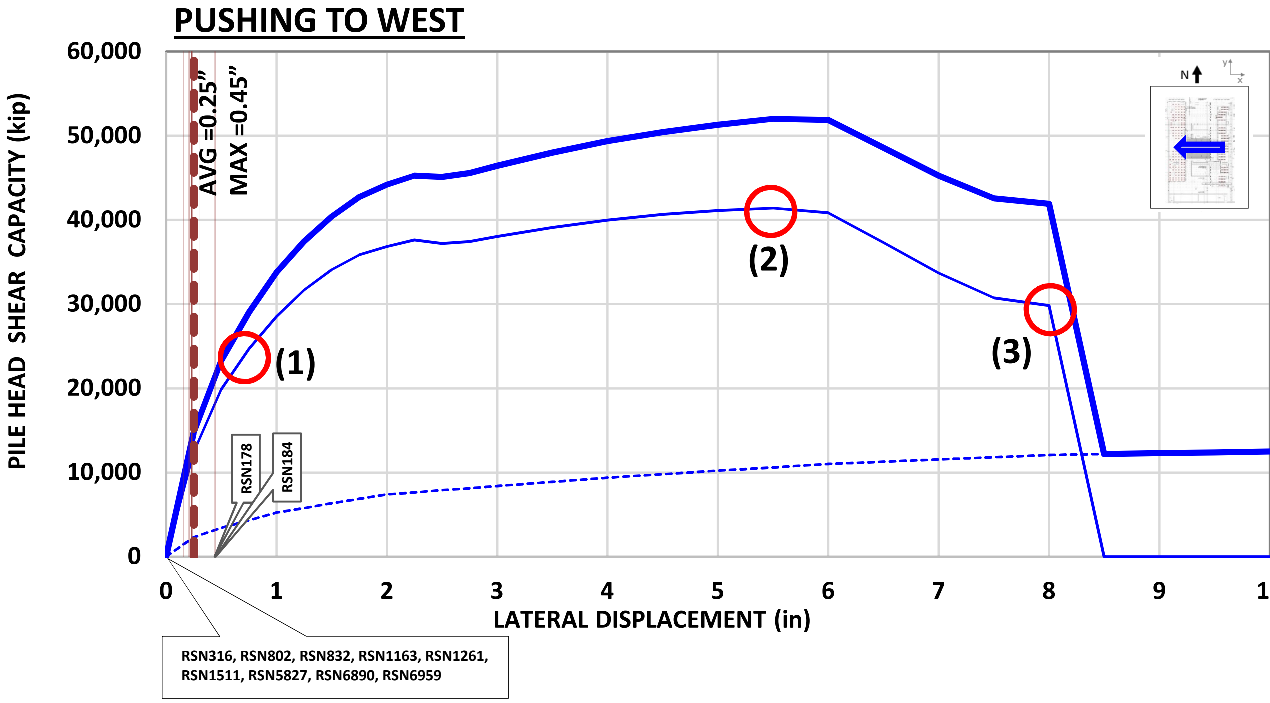
PUSHING TO SOUTH

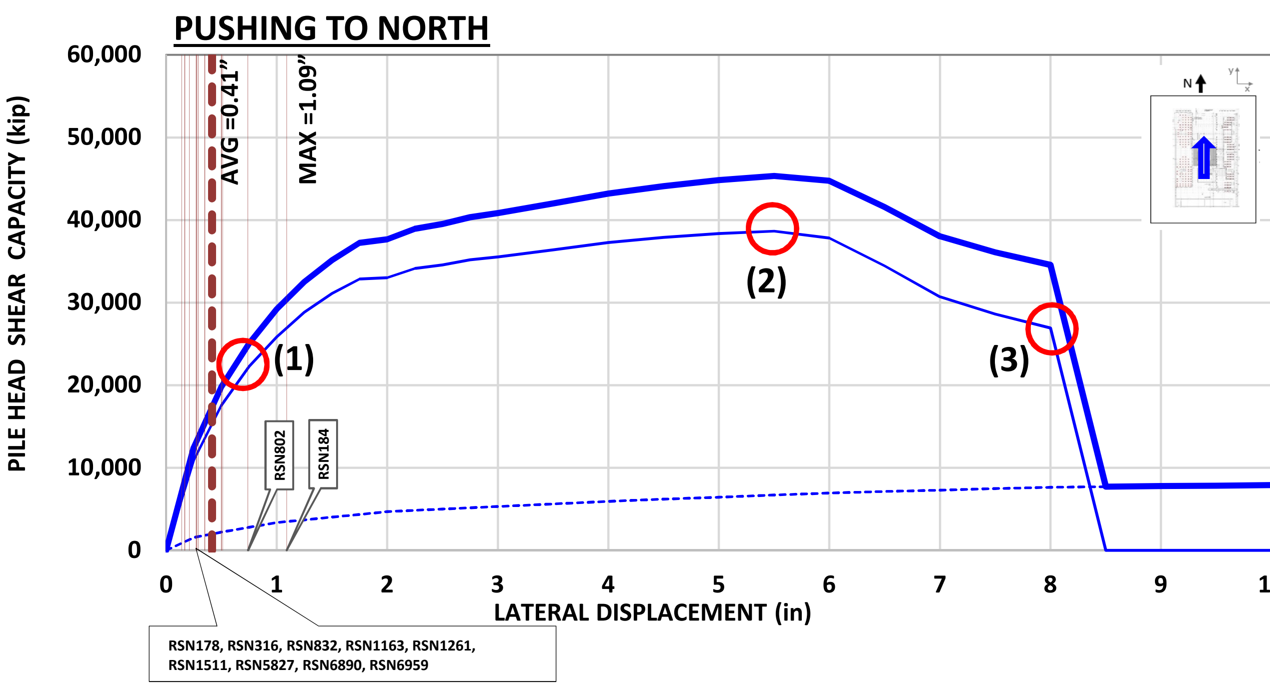


NOTES:

1) Key: (1) First pile yields at its connection to the mat.
(2) First pile experiences significant (>50%) strength degradation.
(3) Second hinge forms in the critical pile.

Figure 25. BSE-1N (2/3*MCE) NLRHA Results



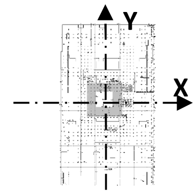


NOTES:

1) Key: (1) First pile yields at its connection to the mat.
(2) First pile experiences significant (>50%) strength degradation.
(3) Second hinge forms in the critical pile.

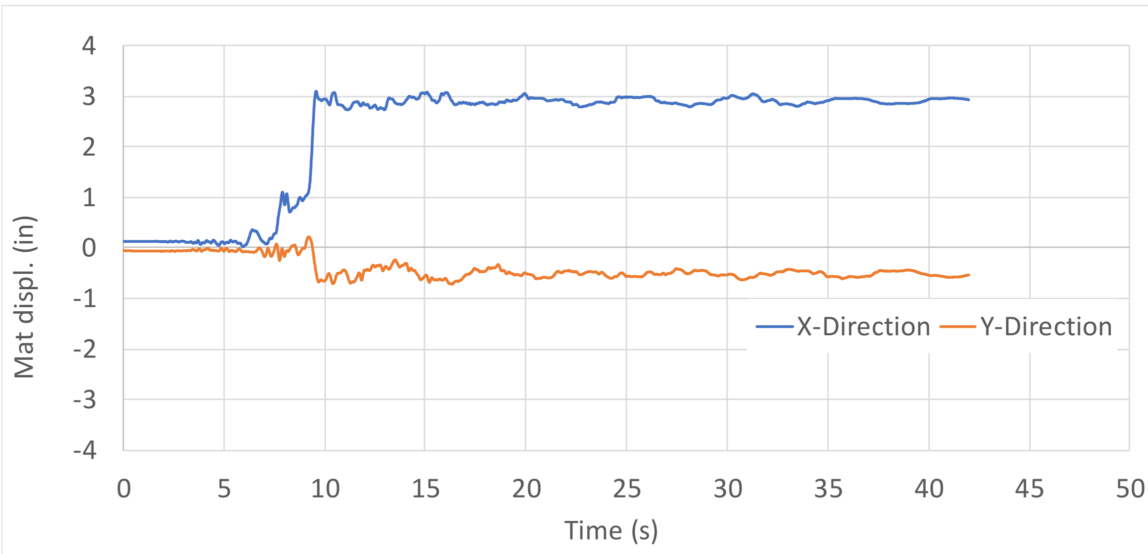
Figure 26. BSE-1N (2/3*MCE) NLRHA Results

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

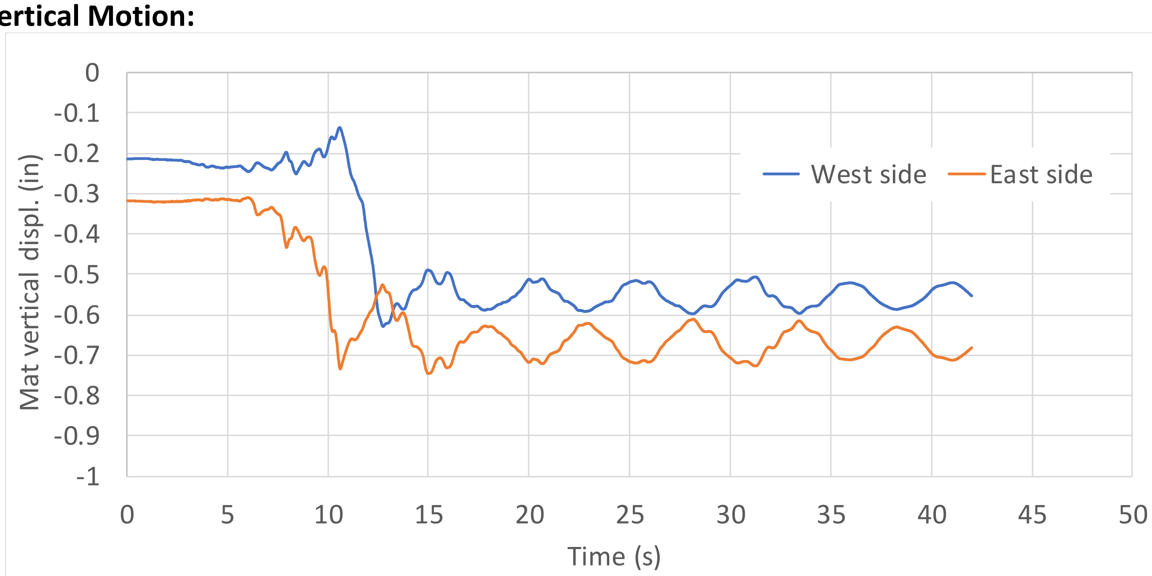


RSN316 WESMORL – Representative of AVERAGE Motion:

Horizontal Motion:



Vertical Motion:

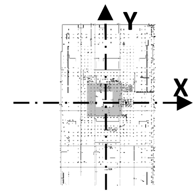


Observations:

- Generally “Limited” permanent displacement observed in most ground motions consistent with Life Safety performance criteria

Figure 27. BSE-1N (2/3*MCE) NLRHA Results – Permanent Deformations

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity



Base Shear:

Ground Motion	Fx (kip)		Fy (kip)	
	Pushing East	Pushing West	Pushing North	Pushing South
RSN178_IMPVALL	21,673	27,472	26,139	21,209
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	22,839	27,424	26,487	25,469
RSN1261_CHICHI	NC	NC	NC	NC
RSN1511_CHICHI	23,105	25,890	27,820	23,660
RSN5827_SIERRA	24,689	28,104	28,282	25,713
RSN6890_DARFIELD	22,983	24,702	36,153	27,651
RSN6959_DARFIELD	19,086	18,208	25,322	23,060
***Median of 11 Ground Motions*[1]**	**24,689**[2]	**27,472**	**36,153**	**27,651**[2]

Notes:
1) Considering the ground motions that do not converge, we have chosen the median ground motion response for each direction (6th largest out of 11) to represent the average performance.
2) Base Shears include unbalanced active and seismic earth pressures

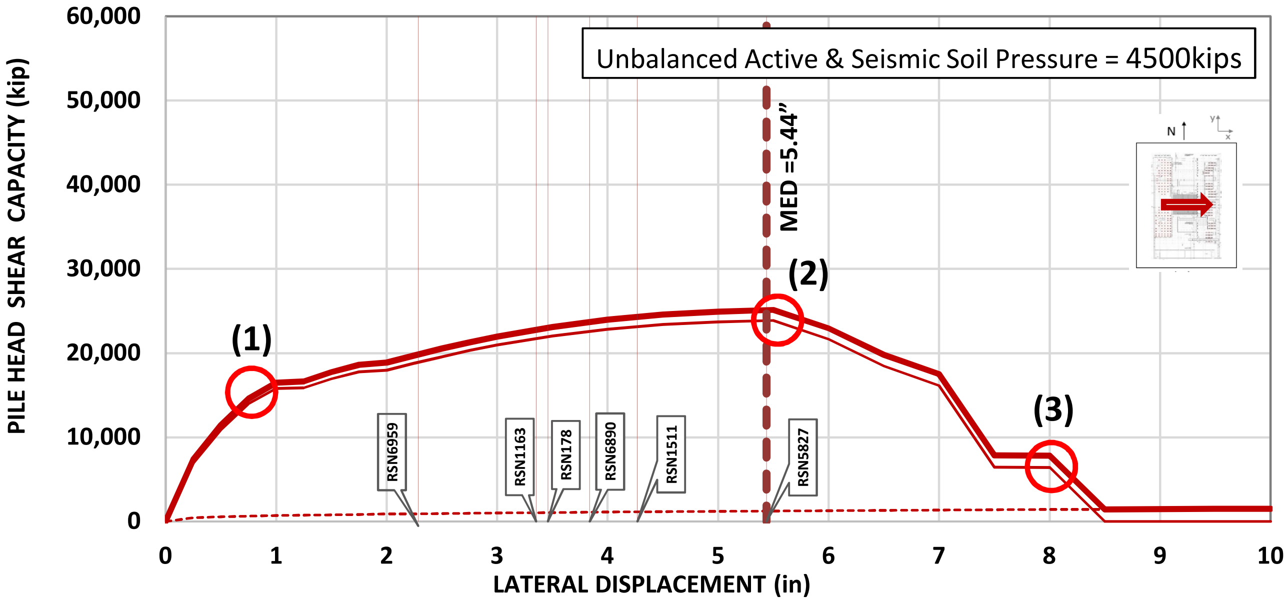
Displacements:

Ground Motion	Maximum Mat Displacement (Average of 4 Corners)				
	Ux (in)		Uy (in)		Uxy (in)
	Pushing East	Pushing West	Pushing North	Pushing South	Resultant
RSN178_IMPVALL	3.46	0.71	0.85	1.42	3.70
RSN184_IMPVALL	NC	NC	NC	NC	NC
RSN316_WESMORL	NC	NC	NC	NC	NC
RSN802_LOMAP	NC	NC	NC	NC	NC
RSN832_LANDERS	NC	NC	NC	NC	NC
RSN1163_KOCAELI	3.35	0.16	0.59	3.34	4.72
RSN1261_CHICHI	NC	NC	NC	NC	NC
RSN1511_CHICHI	4.27	0.37	0.25	2.56	4.88
RSN5827_SIERRA	5.44	0.21	0.21	3.42	6.25
RSN6890_DARFIELD	3.84	0.23	0.59	3.67	4.79
RSN6959_DARFIELD	2.29	0.18	0.28	2.07	2.99
***Median of 11 Ground Motions*[1]**	***5.44***	***0.71***	***0.85***	***3.67***	***6.25***

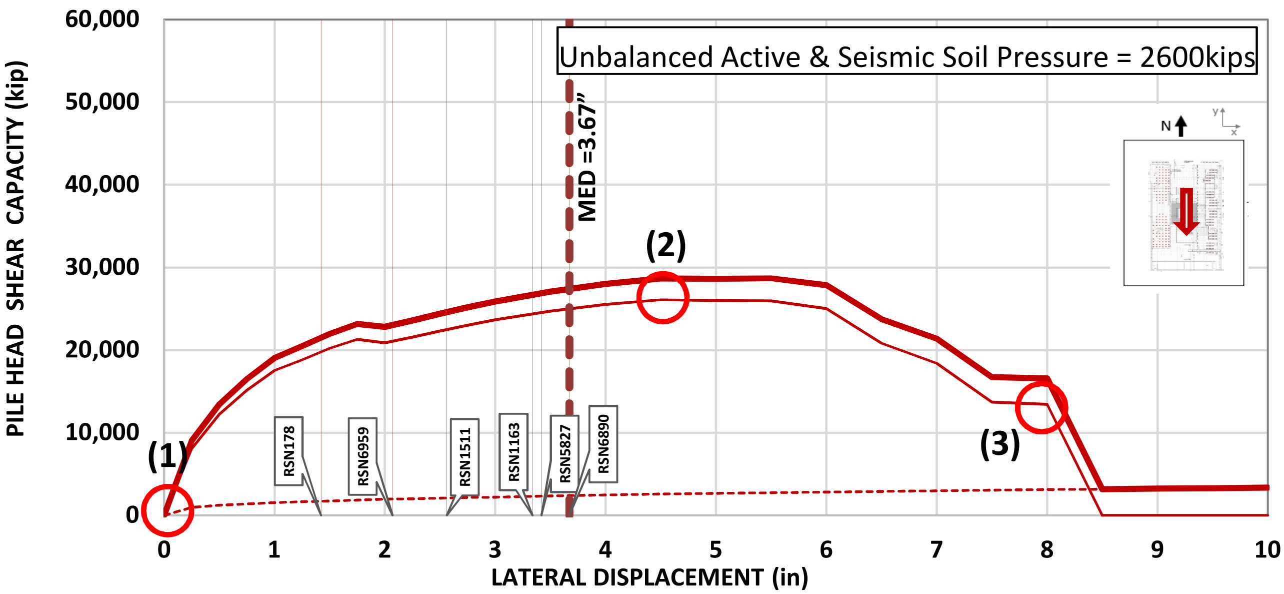
Figure 28. BSE-2N (MCE_R) NLRHA Envelope Results – Base Shear and Displacements

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

PUSHING TO EAST



PUSHING TO SOUTH



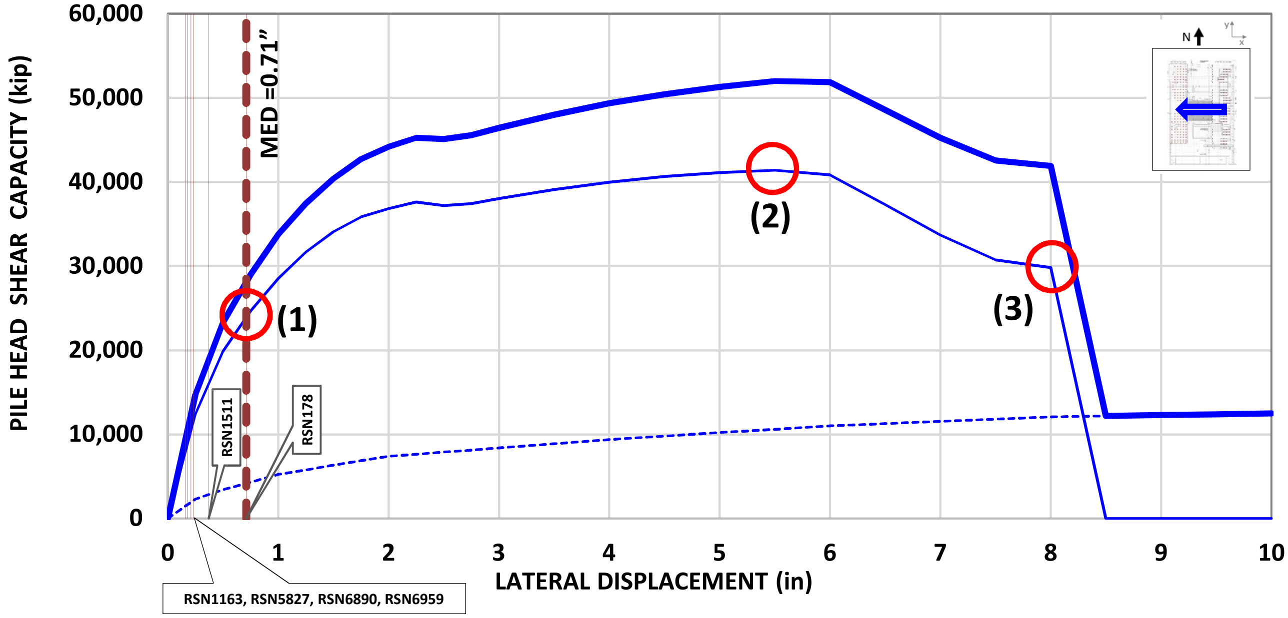
NOTES:

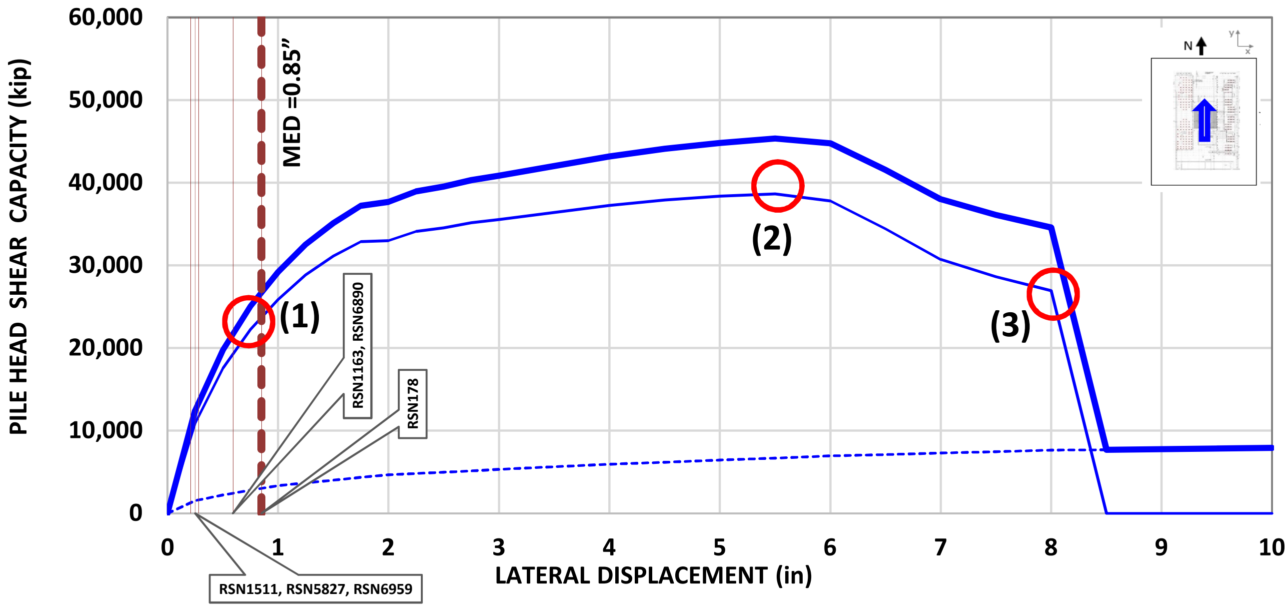
1) 5 of the 11 ground motions (RSN184, 316, 802, 832, and 1261) failed to converge and are omitted from the plots above
2) Key: (1) First pile yields at its connection to the mat.
(2) First pile experiences significant (>50%) strength degradation.
(3) Second hinge forms in the critical pile.

Figure 29. BSE-2N (MCE_R) NLRHA Results

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

PUSHING TO WEST





NOTES:

1) 5 of the 11 ground motions (RSN184, 316, 802, 832, and 1261) failed to converge and are omitted from the plots above
2) Key: (1) First pile yields at its connection to the mat.
   (2) First pile experiences significant (>50%) strength degradation.
   (3) Second hinge forms in the critical pile.

Figure 30. BSE-2N (MCE_R) NLRHA Results