301 MISSION ST. FOUNDATION STABILIZATION

STRUCTURAL BASIS OF DESIGN

September 20, 2018

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1 PROJECT SUMMARY

1.1. Building Description and Location

1.1.1. General

301 Mission Street (also known as the Millennium Tower) is an existing high-rise residential building in San Francisco, California, located on Mission Street, between Freemont Street and Beale Street. The development is comprised of two independent, although functionally connected, structures. One is a 58-story reinforced concrete tower with a single basement and adjacent to that is a 12-story mid-rise residential and amenity building that has five levels of below grade parking below it. A site plan is provide in Figure 1-1 and a section through the site looking south is provided in Figure 1-2. The two buildings are structurally isolated from each other by a continuous expansion joint.

Since their completion in 2009, both structures have experienced continued settlement. The scope of this report includes the foundation retrofit design for the tower structure only.

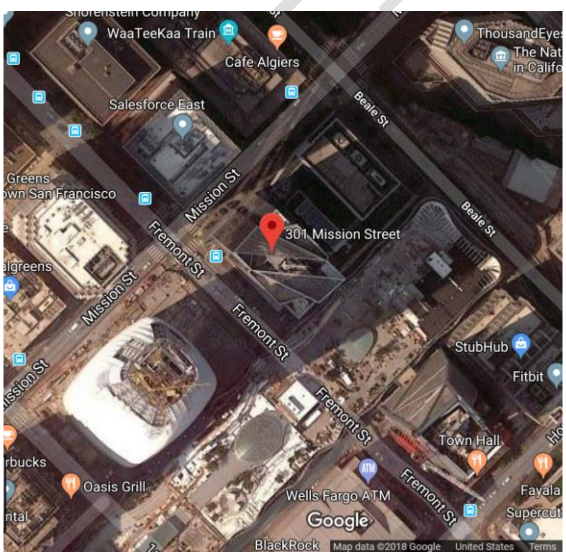


Figure 1-1. Site Plan

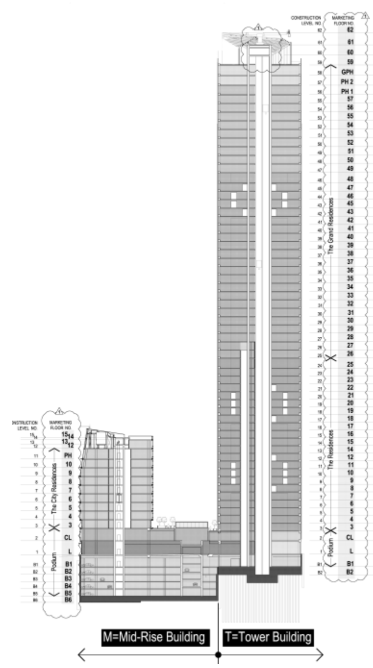


Figure 1-2. Overall Building Section

1.1.2. Description of Existing Structural Systems

1.1.2.1. Superstructure

The tower floors are constructed of flat plate post-tensioned slabs. The floors are supported by a reinforced concrete core and perimeter concrete moment frames. A system of outriggers provides additional lateral resistance in the east-west direction. See Figure 1-3 and Figure 1-4 for illustrations of the tower's primary structural systems and their arrangement.

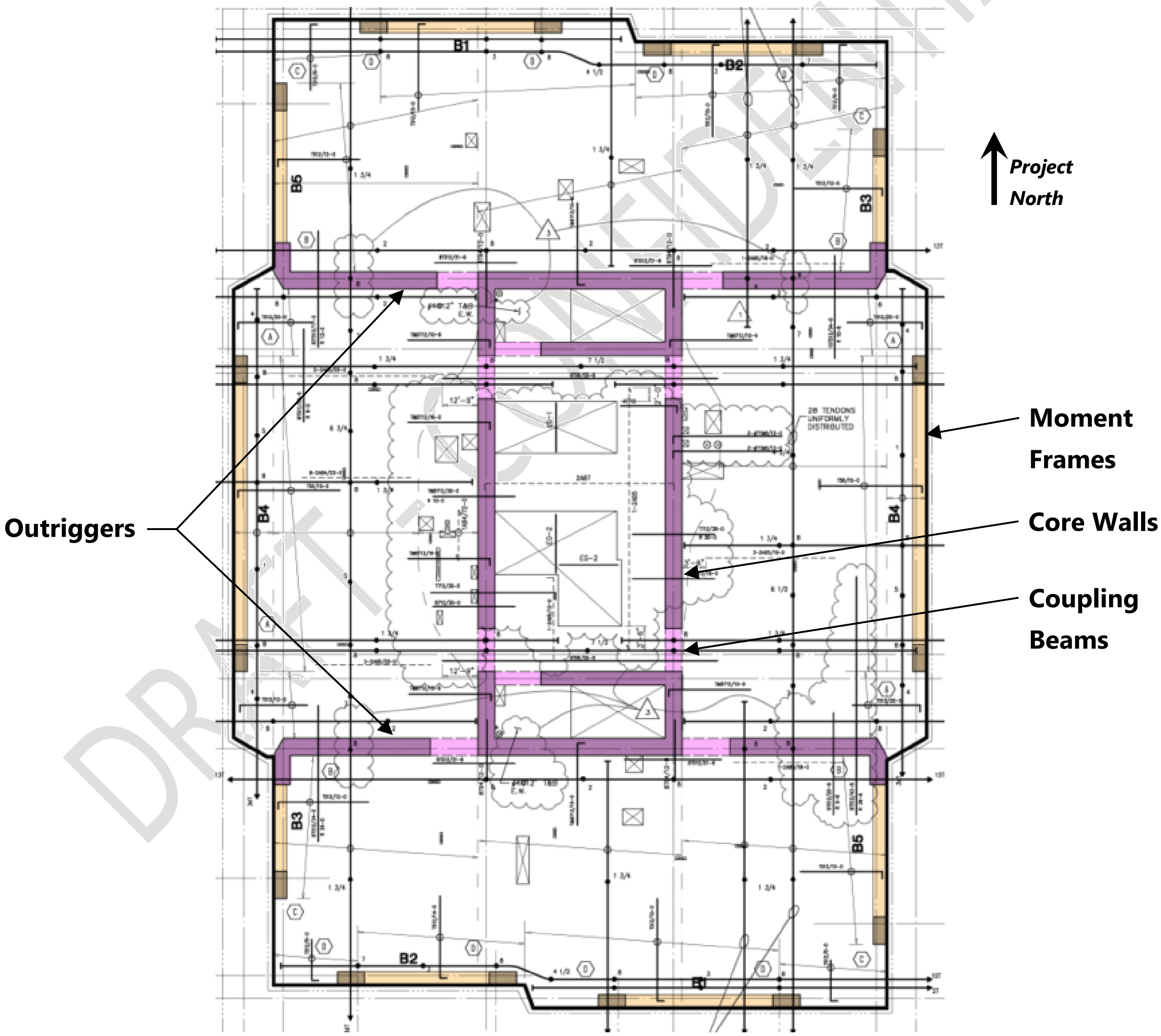


Figure 1-3. Typical Tower Floor Plan (Source: Original Design Drawing S2-1.42.01)

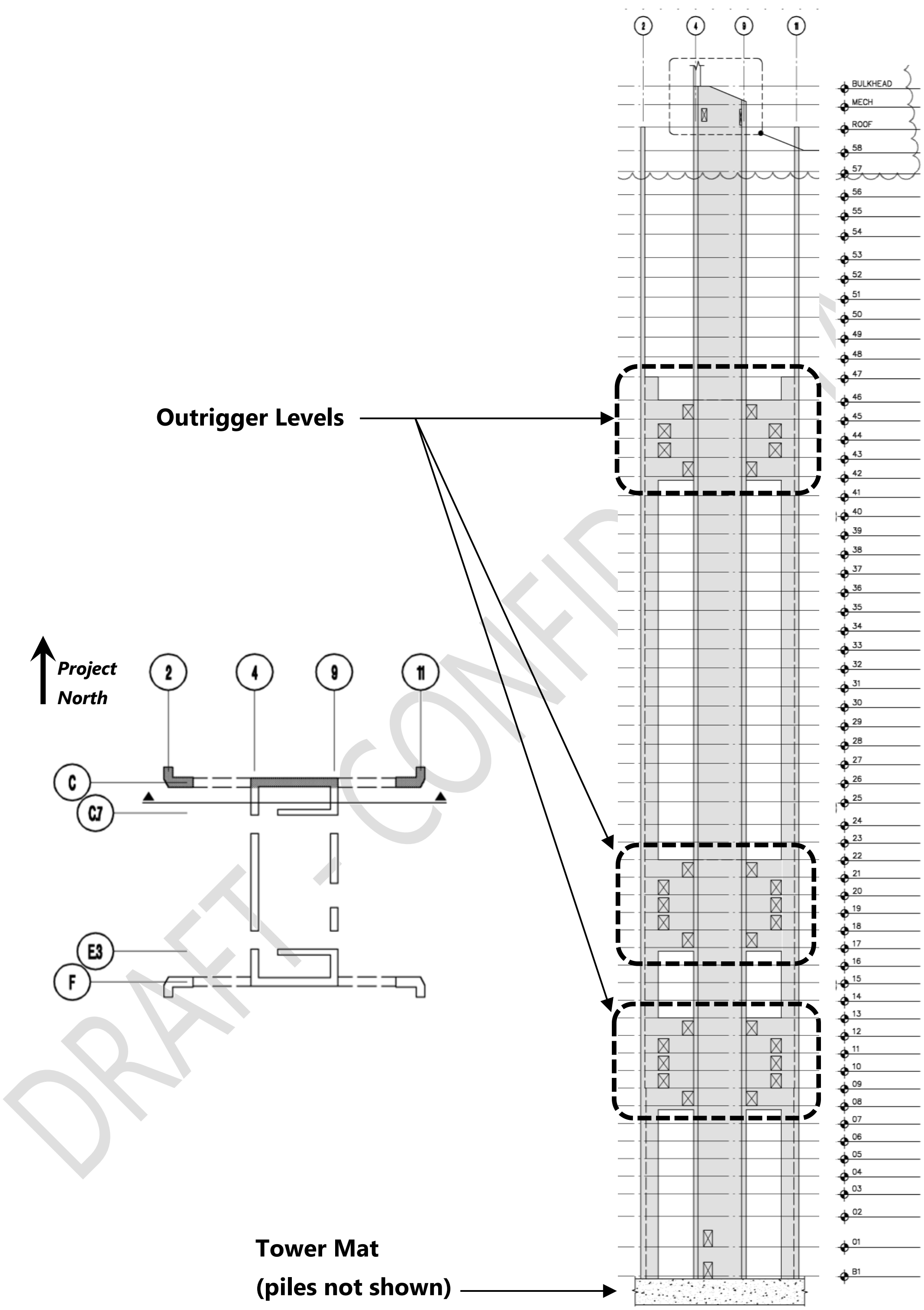


Figure 1-4. Tower Elevation at Core / Outrigger Line C (Line F is Similar) - (Source: Original Design Drawing S3-2.11)

1.1.2.2. Tower Foundation

The tower foundation system is a pile supported mat. It consists of a 10 ft thick reinforced concrete mat supported by nine hundred and thirty-eight 14 in. square prestressed concrete piles that extend into the dense sand Colma formation at depths that vary from approximately 50 to 90 ft below surrounding grade. The key elements of the existing tower foundation are identified in Figure 1-5.

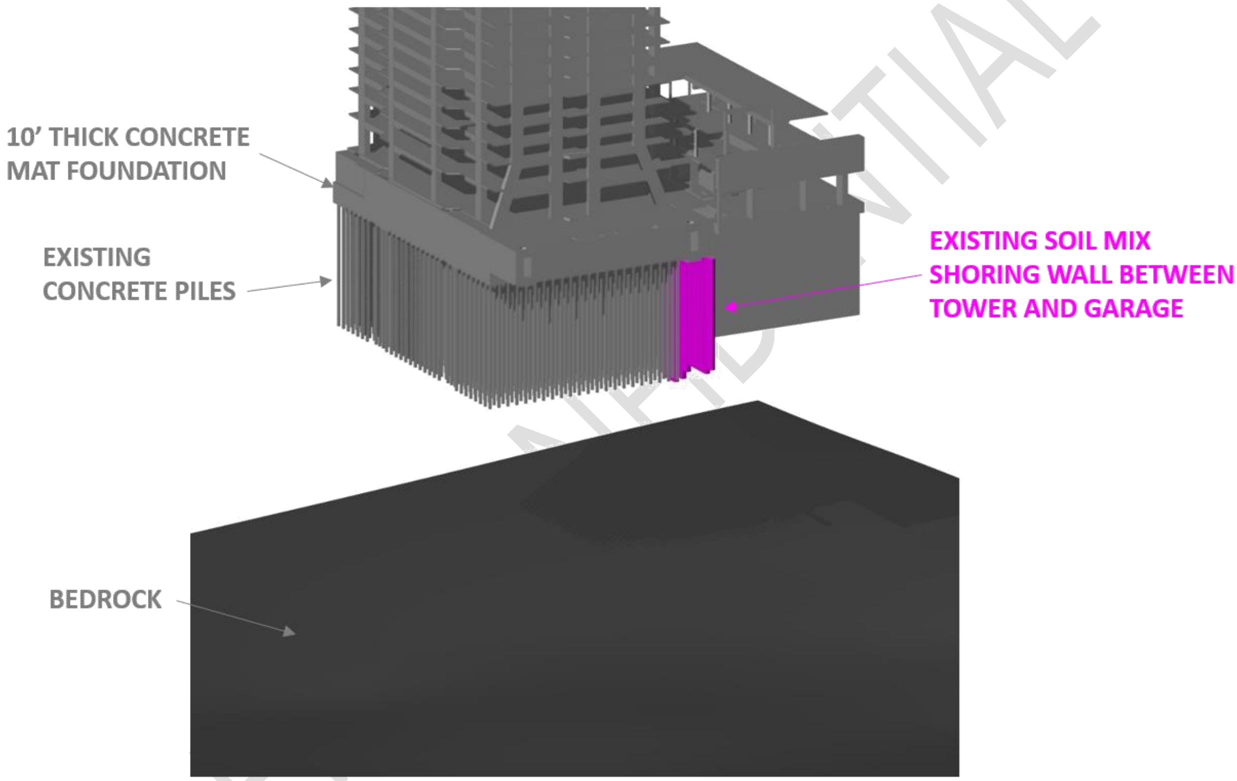


Figure 1-5. Key Foundation Elements

The original design documents specified 945 piles, however 9 piles were broken during the pile driving operation and 2 replacements were driven, resulting in 938 functional piles supporting the tower mat. The piles are typically spaced at either 3′-6″ or 4′-8″ on center with the tighter spacing found beneath the core and under the four outrigger columns. The pile driving record is provided for reference in Appendix B. Figure 1-6 shows the pile arrangement documented in the original design.

As mentioned above, the tower mat is typically 10' deep with two important exceptions. The first is the area in the core where the mat was thickened to 21' deep for a typically 10' wide zone around the necessary elevator pit depressions. This zone is noted on Figure 1-6 and a section through this thickened area is provided in Figure 1-7.

The other exception is the portion of the mat that extends beyond the tower footprint on the south side. The mat in this area was reduced to 3' thick and designed to cantilever from the 10' thick tower mat. No pile support was provided under this portion of the mat. This area is identified on Figure 1-6.

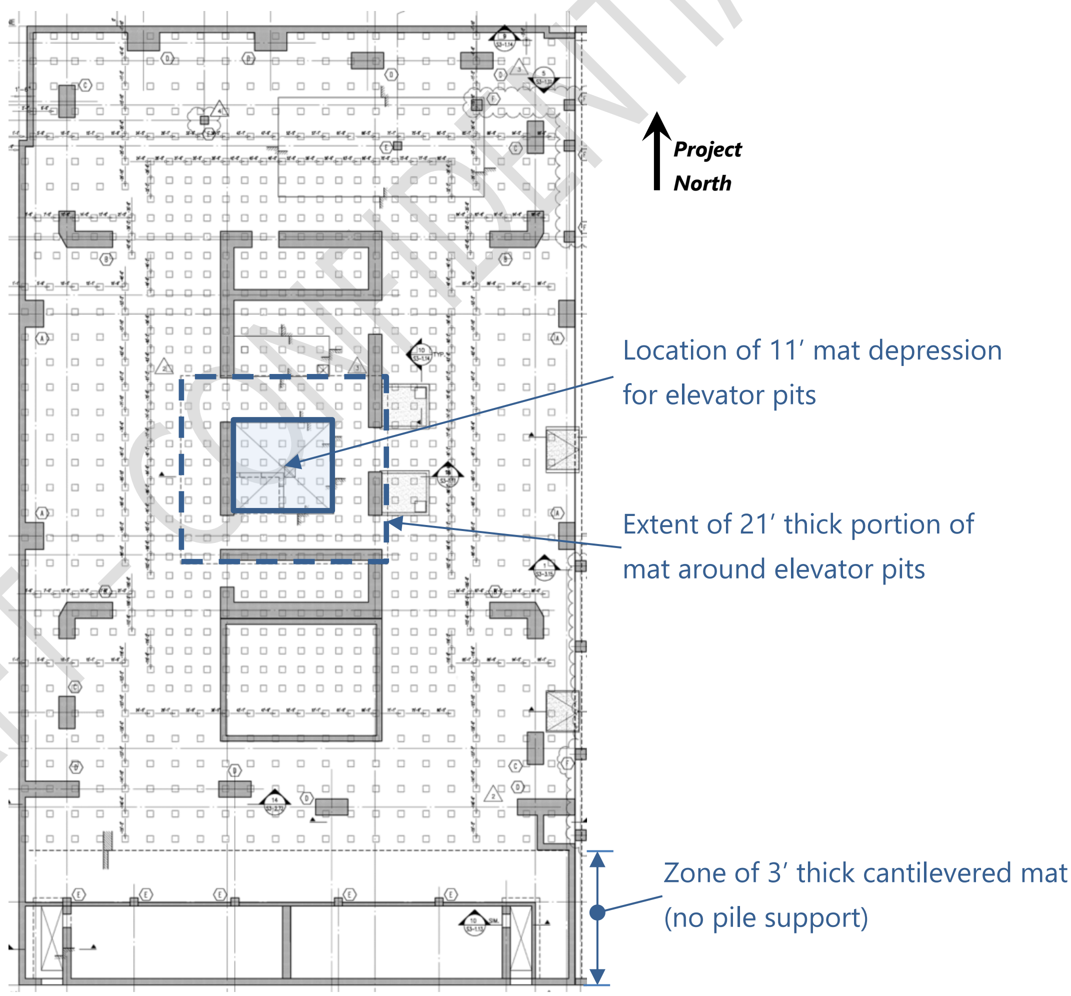


Figure 1-6. Pile Arrangement under Tower Mat per Original Design Drawing S2-0.B1.14

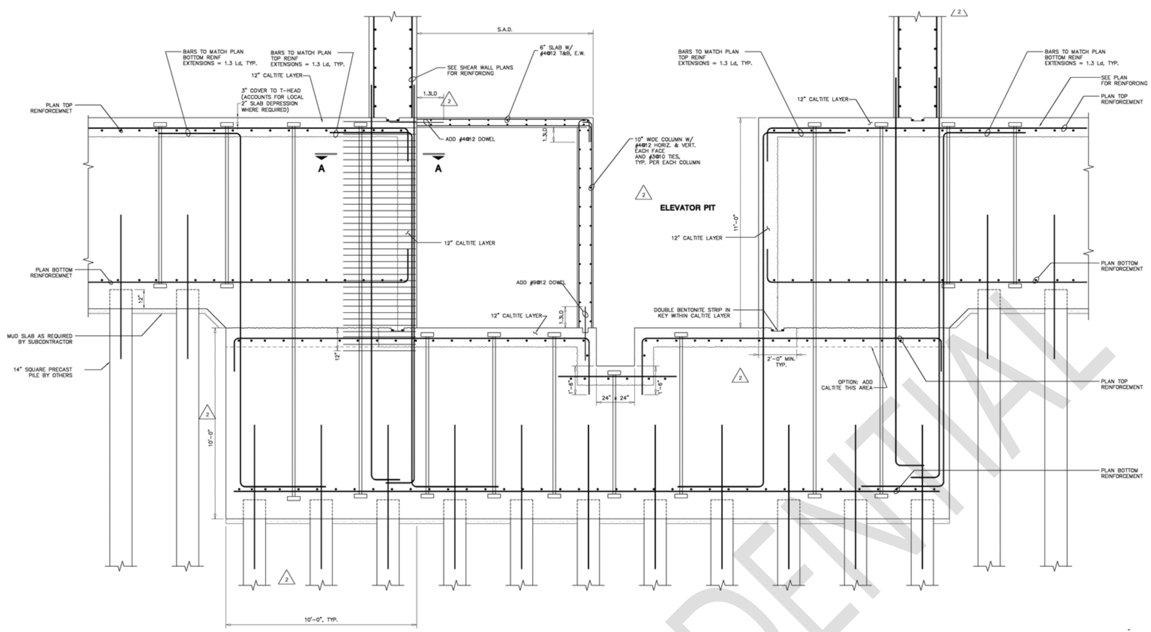


Figure 1-7. Section at Tower Mat Elevator Pits per Original Design Drawing S3-1.11

The tower mat is reinforced with layers of orthogonal longitudinal steel reinforcing bars at the top and bottom of the mat as well as an array of vertical headed steel reinforcing bars spaced at 24” or 36” on center that provide shear resistance. The mat reinforcing layout is provided on original building drawings S2.0.B1.11 – S2.0.B1.13 which have been included in Appendix B for reference. Dowels were provided at the top of the precast concrete piles and cast in to the mat to integrally connect the elements.

1.2. Existing Conditions

1.2.1. Current Settlements and Tilt

The settlement and tilt data at the time of writing this report show that the tower has settled approximately 17” at the lowest point and the top of the building is currently tilting by approximately 16” to the west and approximately 6” to the north. When this document quantifies “tilt,” it is referring to the deviation from vertical as measured at the top of the tower. The northern tilt has remained relatively constant for the past year, but differential settlement across the mat is causing the building to tilt more towards the west as time goes on.

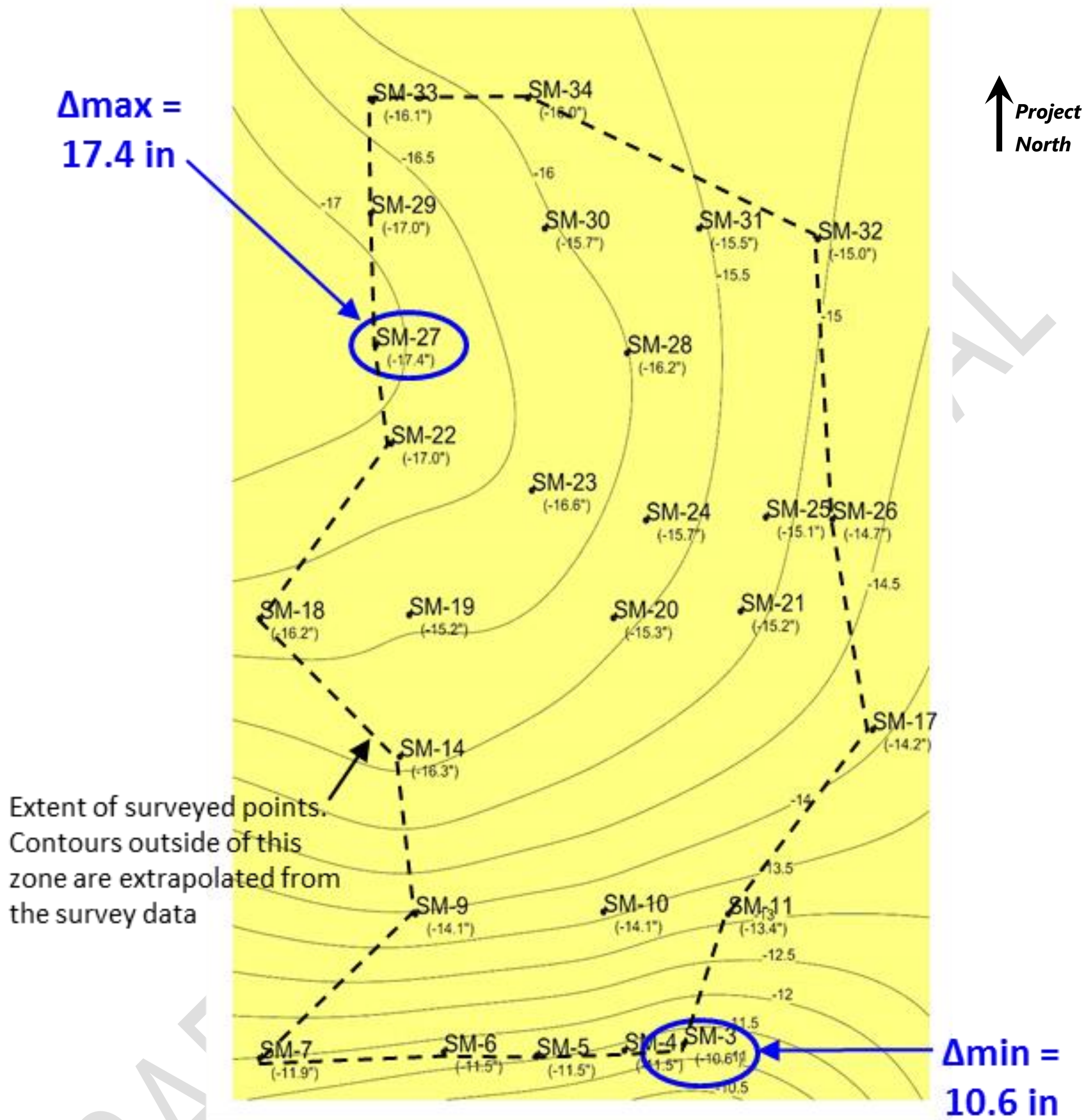


Figure 1-8. Top of Mat Delta from Theoretical Elevation (in) per July 15, 2018 Survey by Langan Engineering and Environmental Services, Inc.

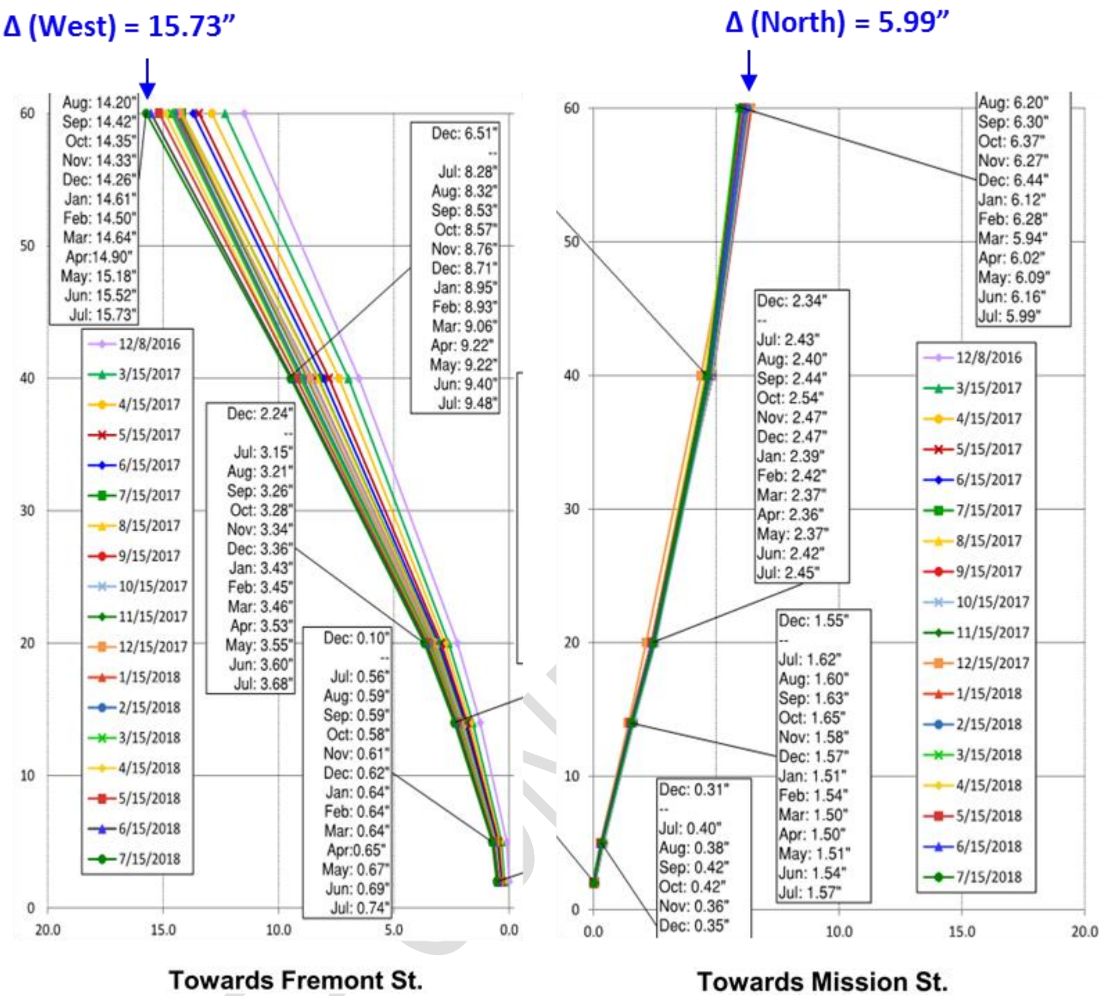


Figure 1-9. Tower offset from plumb (in) per July 15, 2018 Survey by Langan Engineering and Environmental Services, Inc.

1.2.2. Review of Project Record

The record of select original construction records, photos, RFIs, and other project correspondence were reviewed to determine what, if any, changes from the original construction documents were made during the erection of the structure. Significant findings that have impacted the design and analysis of the retrofit are summarized in Appendix B.

2 CODES & REFERENCES

2.1. Controlling Codes, Standards, and Other References

2.1.1. Original Building Design

According to the Foundation Permit Submittal dated 24 May, 2005 the original building was designed in conformance with the 2001 San Francisco Building Code.

During the peer review process it was agreed that a beyond code level capacity design approach should be used for the tower foundation seismic design. These forces were capped by the over-strength load combination forces.

2.1.2. Basis for Providing Compliance with Current Codes

First and foremost, a retrofit of any sort to the foundation is a major structural alteration that must be compliant with the current Building Code.

Selecting and proportioning a reliable retrofit starts with an understanding of the shortcomings present in the existing structure. These shortcomings, which have manifested themselves as excessive settlement and tilt of the existing structure, include the following:

1. Forces delivered to the old bay clay by foundation piling exceed historical pre-compression values.

2. Allowable piling forces specified in the original drawings, and presented to the City and its peer reviewers as evidence of code compliance, were not met for gravity loads, gravity plus wind, or gravity plus seismic loads. See Appendix D for a summary of calculated non-compliance against specified allowable pile forces presented to the City.

This alteration needs to be a retrofit to the existing foundation that reliably bounds stiffness and performance, and accomplishes the following:

- arrests settlement by transferring some or all of the building loads to bedrock,

- eliminates uncertainty by bringing reliability to the building's foundation performance, and
- To the extent the existing foundation is called upon to permanently participate in building support, the Old Bay Clay under the entire building footprint must be at a state of relatively uniform stress that is well below its historical preconsolidation pressure.

Given that the foundation requires a major structural alteration, and with the background that the foundation has not performed as originally predicted for review by the City, it is necessary that the foundation retrofit meet current Building Code requirements.[AJK1]

2.1.3. Foundation Stabilization Retrofit Design

The structural design of the foundation stabilization shall conform with the guidelines for alterations to existing buildings in the 2016 California Existing Building Code with San Francisco Amendments. Per Section 403, alterations to any building shall comply with the requirements of the 2016 San Francisco Building Code.

The 2016 San Francisco Building Code, hereby referred to as the Building Code, is comprised of the 2016 California Building Code (CBC) as amended by the 2016 San Francisco Building Code Amendments. The Building Code references the following standards:

- ASCE 7-10, *Minimum Designs Loads for Buildings and Other Structures*, by the American Society of Civil Engineers
- ANSI/AISC 360-10, *Specification for Structural Steel Buildings*, by the American Institute of Steel Construction
- ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, by the American Institute of Steel Construction
- ACI 318-14, *Building Code Requirements for Structural Concrete*, by the American Concrete Institute

The Building Code requires that the tower stabilization meet the Chapter 16 prescriptive code structural requirements and Chapter 18 foundation design requirements.

Additionally, consistent with the current practice for tall building design, the foundation retrofit seismic design will meet the performance-based requirements described in San

Francisco Administrative Bulletin AB-083 *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures*, hereafter referred to as AB-083, and the recommendations of PEER *Guidelines for Performance-Based Seismic Design of Tall Buildings,* Version 2.03, PEER 2017/06, hereafter referred to as the PEER *Guidelines*.

In accordance with Section 4.1 of AB-083, requested Code exceptions to the prescriptive seismic requirements are listed in the following section. It is our intent that the performance based design approach satisfies the beyond code level requirement that was required in the original design.

Additionally, as the building is to remain occupied throughout construction, interim checks against a selection of the Building Code prescriptive requirements and PEER Guidelines performance-based requirements are being performed at key milestones as discussed in Section 7 to verify building safety is maintained throughout construction.

Other references include:

- *Geotechnical Memorandum – 301 Mission Retrofit Design* dated 13 April, 2018 by ENGEO Incorporated
- Original Building Structural Design Drawings by DeSimone Consulting Engineers (See Appendix B for drawing list)
- *Foundation Permit Submittal Volume I - IV* dated 24 May, 2005 by DeSimone Consulting Engineers
- *Revised Geotechnical Investigation – 301 Mission Street* dated 13 January, 2005 by Treadwell & Rollo

2.2. Exceptions to Building Code Provisions

The following enhancements to the Building Code have been applied to the design of the retrofit:

- The overstrength factor, Ω_0, in ASCE 7-10 Table 12.2-1 and associated load combinations in ASCE 7-10 §12.4.3 will not be used for the design of the foundation stabilization. Instead, to be consistent with the use of performance-based design, the MCE_R demands from the NLRHA will be used in conjunction with the force-controlled action recommendations in the PEER *Guidelines* to proportion the critical foundation elements which are traditionally governed by load combinations with overstrength factor. It is our intent that this approach is consistent with the beyond code level requirement included in the original foundation design.

3 RETROFIT DESCRIPTION

The foundation stabilization retrofit consists of the following foundation types:

1. New piles to rock to arrest settlement and resist a portion of gravity and seismic load (approximately 132 total).
2. Existing piles to sand to carry reduced gravity loads and seismic loads.

The proposed retrofit will be installed as described below:

PROPOSED CONSTRUCTION SEQUENCE:

STEP 1: DEMO AND RELOCATE B1 LEVEL NON-STRUCTURAL ELEMENTS AS REQUIRED IN THE ARCHITECTURAL DRAWINGS.

STEP 2: INSTALL 132 PILES TO ROCK ON THE WEST AND EAST SIDE OF THE TOWER B1 LEVEL. PRELOAD PILES IN ACCORDANCE WITH THE STRUCTURAL DRAWINGS AND CONNECT PILES TO MAT.

STEP 3: REBUILD B1 LEVEL.

Figure 3-1. Retrofit Construction Sequence

The new piles will be installed in between existing piles in the arrangement shown in Figure 3-2. The new piles will be displacement piles and are comprised of a 13 5/8″ diameter outer steel casing and a 9 5/8″ diameter inner steel casing. The outer casing extends down 30 feet (±) below the bottom of the mat to provide improved bending performance. The inner casing is extended through the Old Bay Clay and socketed approximately 60′ into the bedrock below. Figure 3-3 shows the upper and lower rock pile sections.

See the Geotechnical Report for more information regarding the pile installation method.

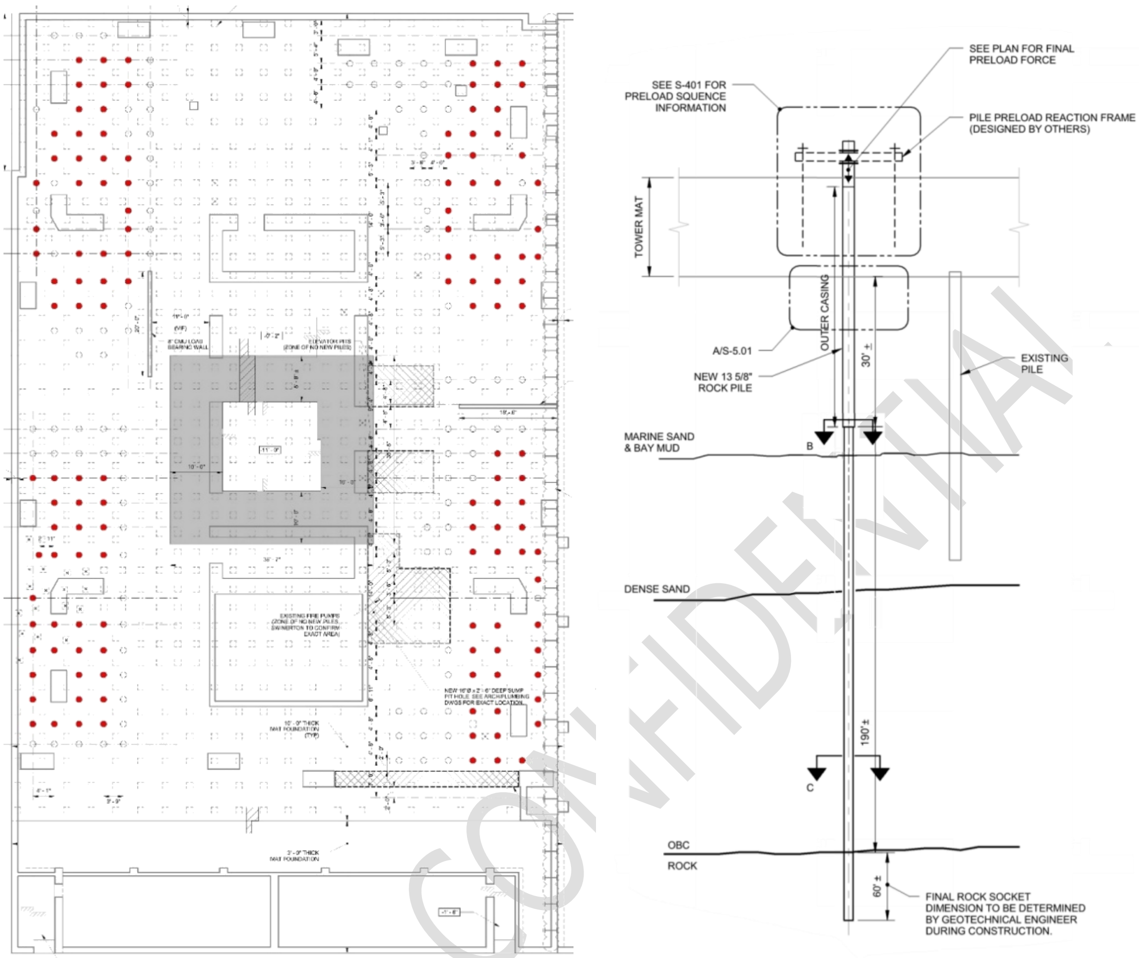


Figure 3-2. Retrofit Plan and Rock Pile Elevation

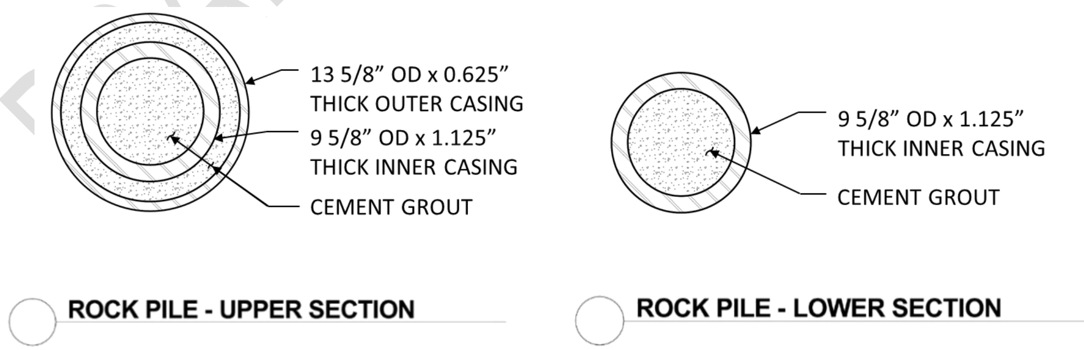


Figure 3-3. Rock Pile Sections

4 PERFORMANCE OBJECTIVES

4.1. Global Building Performance

4.1.1. Code Prescriptive Gravity

The retrofit shall demonstrate that it reliably arrests future settlement due to sustained gravity loads while improving (or having negligible impact on) the current building performance during seismic events.

The retrofit design shall be proportioned to meet the California Building Code minimum requirements for allowable loads on foundation elements with Code required factors of safety.

4.1.2. Code Prescriptive Lateral (Wind and DBE/R)

Wind

According to Section 2.3.2 of the original Foundation Permit Submittal, the original building was designed for wind forces determined by a wind tunnel simulation performed by Rowan Williams Davies & Irwin Inc. (RWDI). LERA did not participate in this wind study, and the wind environment has changed with the addition of new high rises nearby. In lieu of a new wind study that considers the current site conditions, the retrofit shall demonstrate that it conforms to the prescriptive 2016 California Building Code requirements for wind loads. Note that the structure will be reviewed for adequate serviceability performance against code prescriptive wind forces. Human comfort checks will not be considered.

The wind performance objectives for the design of the retrofit are as follows:

Table 4-1. Wind performance objectives

Level of wind event	Performance objectives
Ultimate (700-year) design event	Structure remains essentially elastic.

DBE

The DBE performance objectives for the design of the retrofit are as follows:

Table 4-2. Code prescriptive seismic performance objectives

Level of earthquake	Performance objectives
Design basis earthquake (DBE)	• Low risk of life-threatening injury from structural and nonstructural damage. • Extensive structural and nonstructural damage may occur. Repairs may be required prior to reoccupation and may not always be economically feasible.

4.1.3. Performance Based Seismic (SLE and MCE)

The performance-based procedure in the PEER *Guidelines* is used to design the retrofit, as permitted by the Building Code §104.11 *alternate design* clause, the ASCE 7-10 §1.3.1.3 *performance-based procedures* clause.

The design is intended to achieve the performance objectives stated in ASCE 7-10 and provide a level of safety and ductility equivalent to that provided by a prescriptive design in accordance with the Building Code.

The seismic performance objectives for the design of the building are shown in Table 4-3. The performance objectives are adopted from the PEER *Guidelines* and ASCE 41-13.

Table 4-3. Seismic performance objectives

Level of earthquake	Performance objectives
Service level earthquake (SLE) 50% probability of being exceeded in 30 years (43-year return)	Structure remains essentially elastic with calculated deformations less than those that results in damage that: - exceeds minor cracking of concrete or yielding of steel in a limited number of structural elements, - impairs the ability of the structure to survive MCER shaking, - results in unacceptable permanent deformation, or - requires repairs beyond that which is necessary to restore appearance or protection from water intrusion, fire, or corrosion.
Risk-targeted maximum considered earthquake (MCE_R)	• Low probability of collapse (10% probability or less), otherwise known as collapse prevention. • Substantial structural and nonstructural damage is expected. Extensive repairs are required prior to reoccupation and may not be economically feasible.

4.2. Component Performance Objectives and Classification

4.2.1. Code Prescriptive Gravity

Individual foundation elements (existing and new piles) are designed to meet code requirements for allowable loads with the minimum Code required factors of safety = 2.0 (per CBC §1810.3.3.1.7).

4.2.2. Code Prescriptive Lateral (Wind and DBE/R)

Individual foundation elements (existing and new piles) are designed to meet code requirements for allowable loads with the minimum Code required factors of safety = 2.0 (per CBC §1810.3.3.1.7) with 1/3 increase per ENGEO recommendations.

4.2.3. Performance Based Seismic (SLE and MCE)

The following elements are designed to yield under seismic loading and are therefore classified as deformation-controlled actions per PEER *Guidelines* §6.8.2:

- axial force in existing piles,
- flexure in existing piles, and
- flexure in new rock piles.

The following actions are designed as ordinary force-controlled actions:

- axial force in new rock piles, and
- shear and flexure in concrete mat.

5 OVERVIEW OF DESIGN PROCEDURE

The design of the foundation stabilization follows a four step design and analysis procedure as described below:

1. **Code Gravity design and foundation settlement analysis** – analyze the existing structure to establish the gravity load distribution in the foundation elements today based on the current foundation settlement pattern. Gravity load distribution is then reviewed at the major construction milestones noted in Section 7.3 to verify the building performance is improving at each interim step. Uncertainties in geotechnical properties (lower, best guess, and upper bound soil properties) are analyzed and accounted for as noted in Section 7.6.2.

   Foundation elements will be reviewed at the completion of the retrofit and at the time when settlement is fully arrested (depending on the soil properties assumed these may or may not happen at the same time) for Code prescribed allowable (for piles) and ultimate (for mat) loads and capacities.

2. **Code Wind and DBE/R design** – design retrofit for Code prescribed wind and DBE/R forces using a generally linear elastic analysis model. Piles are modeled with nonlinear properties to account for the fact that many of the existing piles have initial forces beyond their yield point. Pile and mat foundation initial conditions for each interim construction milestone and soil property assumption are established from the gravity load distribution and foundation settlement analysis.

   a. Code prescribed strength checks will be accomplished.

   b. Initial design of structural elements and actions that are intended to remain essentially elastic at MCE_R will use seismic forces that are amplified based on experience.

3. **Service Level nonlinear response history analysis verification** – verify that the initial design meets the performance objectives for Service Level events at each interim construction milestone using nonlinear response history analysis (NLRHA) with Service Level ground motion records, with pile and mat foundation initial conditions established from the gravity load distribution and foundation settlement analysis.

4. **MCE_R nonlinear response history analysis verification and design** – verify that the initial design meets the performance objectives for MCE_R at each interim construction milestone using nonlinear response history analysis (NLRHA) with MCE_R ground motion

records, with pile and mat foundation initial conditions established from the gravity load distribution and foundation settlement analysis.

a. The initial design is revised as necessary to achieve the performance objectives.

b. Final design of structural elements using the analysis results from the NLRHA.

6 ACCEPTANCE CRITERIA

6.1. Global Acceptance Criteria

The global acceptance criterion for wind loading is listed below.

Table 6-1. Wind global acceptance criteria

Item	Maximum acceptable value
Service-level interstory drift Note 1	h/500

1. Service-level wind loads are defined by the ASD Load Combinations specified in Section 12.5.

The global superstructure acceptance criteria for DBE, SLE and MCE, selected in accordance with the PEER *Guidelines*, are listed below.

Table 6-2. DBE global acceptance criteria

Item	Maximum acceptable value
Design story drift	2%

Table 6-3. SLE global acceptance criteria

Item	Maximum acceptable value
Peak transient drift	0.5%

Table 6-4. MCE global acceptance criteria

Item	Maximum acceptable value
Peak transient drift	3% from the mean response, 4.5% from any individual ground motion
Residual drift	1% from the mean response, 1.5% from any individual ground motion

6.2. Component Acceptance Criteria

6.2.1. Existing Superstructure

Component acceptance criteria for deformation-controlled components under MCE are listed in the following table.

Table 6-5. Component Acceptance Criteria

Component	Maximum acceptable value
Coupling beams	
Composite	0.06 rad (Motter et al. (2017))
Conventional	CP plastic rotation in ASCE 41-17
Core walls	
Confined concrete	Compressive strain of 0.015 (PEER *Guidelines*)
Unconfined concrete	Compressive strain of 0.003 (PEER *Guidelines*)
Reinforcing steel	Tensile strain of 0.05 (ASCE 41-17; PEER *Guidelines*), compressive strain based on confinement detailing
Diagonally-reinforced outriggers	[AJK2]
Moment frame beams	CP plastic rotation in ASCE 41-17
Moment frame columns	CP plastic rotation in ASCE 41-17

6.2.2. Foundation

6.2.2.1. Pile Axial Loads

<u>Gravity Loads</u>
Piles are designed using Allowable Stress Design load combinations for gravity axial loads. Component acceptance criteria for gravity loads are listed in Table 6-6.

Table 6-6. Foundation Component Axial Load Acceptance Criteria - Gravity

<table>
<tr><th>Component</th><th>Maximum acceptable value</th></tr>
<tr><td>Existing Piles</td><td rowspan="2">Allowable axial load, Pa = Ultimate Capacity / 2
(Minimum factor of safety = 2.0 per CBC §1810.3.3.1.7)</td></tr>
<tr><td>New Rock Piles</td></tr>
</table>

<u>Wind and Design Basis Earthquake (DBE)</u>

Piles are designed using Allowable Stress Design load combinations for wind and DBE axial loads. Component acceptance criteria for wind and DBE loads are listed in Table 6-7.

Table 6-7. Foundation Component Axial Load Acceptance Criteria – Wind and DBE

<table>
<tr><th>Component</th><th>Maximum acceptable value</th></tr>
<tr><td>Existing Piles</td><td rowspan="2">Allowable axial load, Pa = 1.33 * Ultimate Capacity / 2
(Minimum factor of safety = 2.0 per CBC §1810.3.3.1.7
with 1/3 increase per ENGEO recommendations)</td></tr>
<tr><td>New Rock Piles</td></tr>
</table>

<u>Service Level Earthquake (SLE)</u>

???

<u>Risk-targeted Maximum Considered Earthquake (MCE_R)</u>

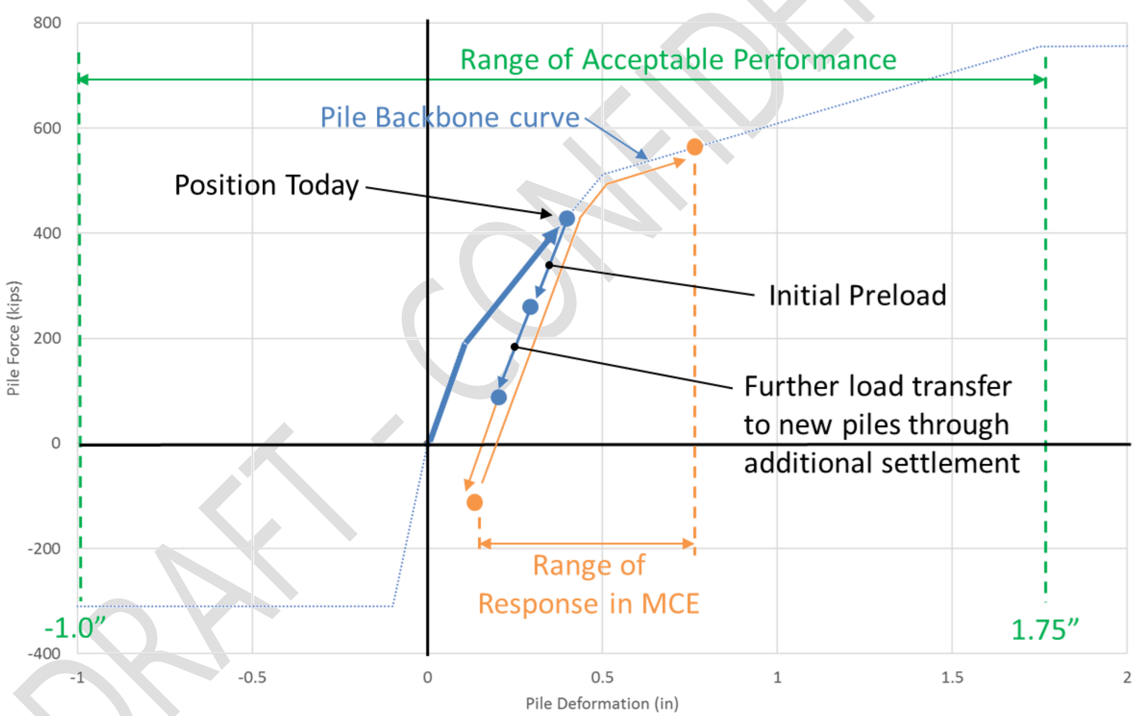
Existing piles are classified as deformation-controlled components under MCE. Rock piles are designed for ordinary force-controlled actions. Component acceptance criteria at MCE are listed in Table 6-8.

Table 6-8. Foundation Component Axial Load Acceptance Criteria - MCE

Component	Maximum acceptable value
Existing Piles	Vertical deformation limits per ENGEO recommendations See Figure 6-1.
New Rock Piles	Pult < 2000 kips (Compression)[1] Pult < 1335 kips (Tension)[2][AJK3]

[1] Rock Pile compression capacity limited to testing load. See Section **Error! Reference source not found.**.

[2] Rock Pile tension capacity limited to tension structural capacity.



EXAMPLE LOAD PATH AT PILE P1 (NW Corner of mat)

Figure 6-1. Existing Pile MCE Vertical Deformation Acceptance Criteria

6.2.2.2. Pile Lateral Loads

Wind and Design Basis Earthquake (DBE)
???

Service Level Earthquake (SLE)
All piles are classified as deformation-controlled components for Service Level lateral loads. Per PEER Guidelines §5.8.1, calculated deformations shall be less than those that result in damage that:

(a) Exceeds minor cracking of concrete or yielding of steel in a limited number of structural elements
(b) Impairs the ability of the structure to survive MCE_R shaking
(c) Results in unacceptable permanent deformation
(d) Requires repairs beyond which is necessary to restore appearance or protection from water intrusion, fire, or corrosion.

To satisfy these requirements, we have adopted the maximum acceptable pile head displacements presented in Table 6-9.

Table 6-9. Foundation Component Lateral Load Acceptance Criteria - SLE

Item	Maximum acceptable value
Peak transient pile head displacement	X″ from the mean response, X″ from any individual ground motion
Residual pile head displacement	X″ from the mean response, X″ from any individual ground motion

Risk-targeted Maximum Considered Earthquake (MCE_R)
All piles are classified as deformation-controlled components for lateral loads under MCE. Since pile backbone curves with ultimate strain limits are explicitly modeled in the MCE NLRH analysis, the design is considered acceptable where the ground motion displacement doesn't cause the pile group to lose its load carrying capacity. See Figure 6-2 to Figure 6-5 for the summation of lateral load-displacement curves for all foundation elements in each orthogonal direction.

Additionally, individual pile head displacements are limited to the values where the piles start to lose load carrying capacity. See Figure 7-33 and Figure 7-38 to observe these limiting values for each pile type and varying axial load.

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

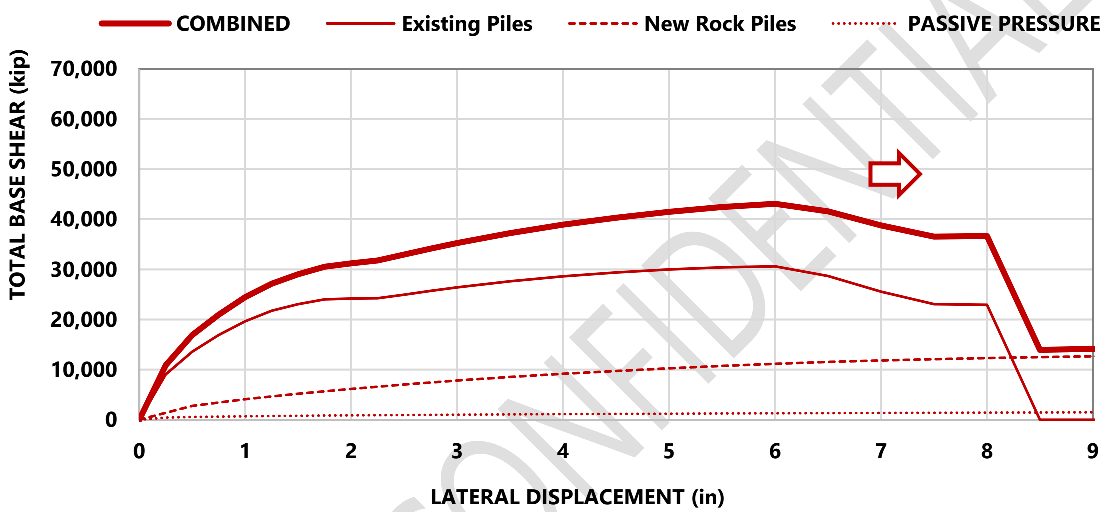


Figure 6-2. Global Foundation Pushover – Pushing East

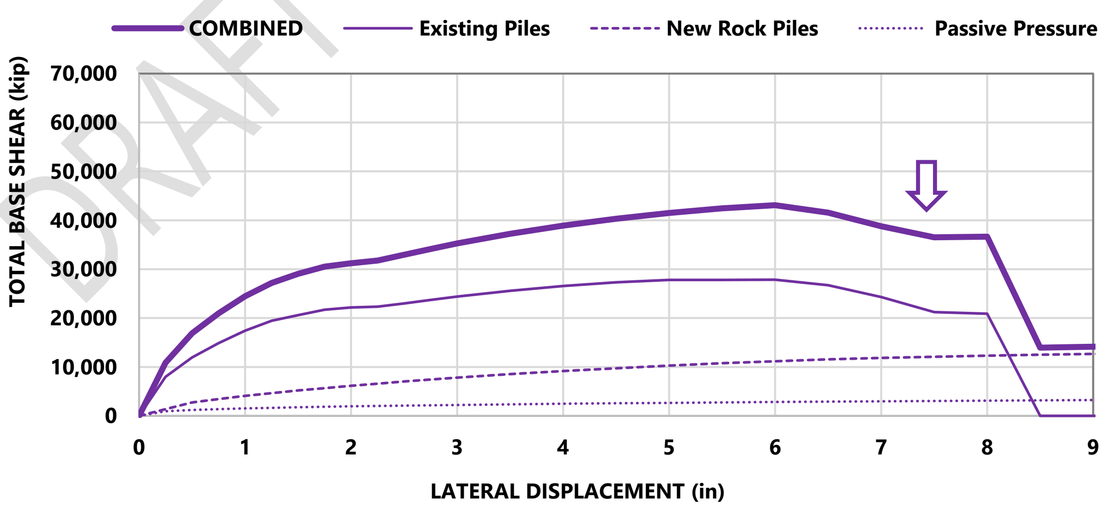


Figure 6-3. Global Foundation Pushover – Pushing South

Pile Lateral Stiffness	Pile Force
Lower Bound	Gravity

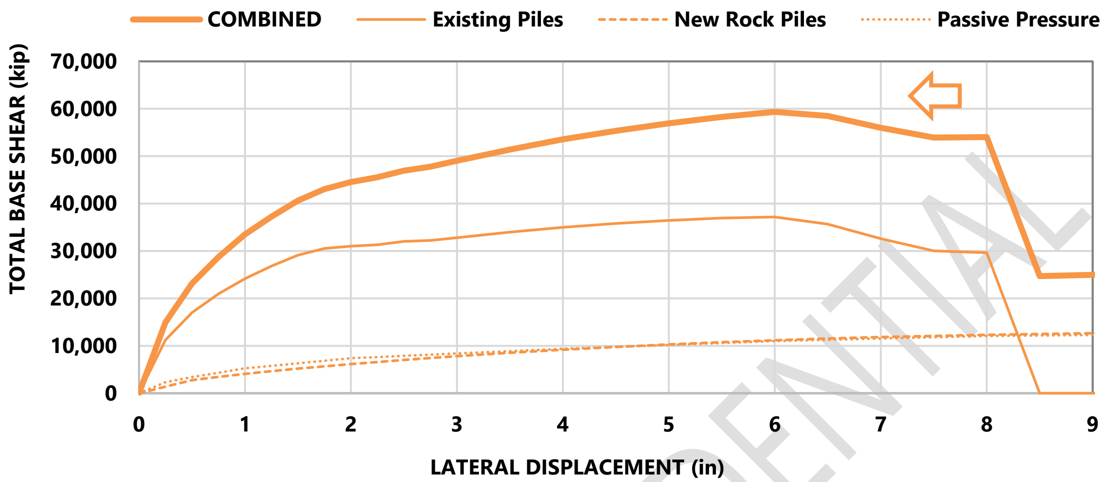


Figure 6-4. Global Foundation Pushover – Pushing West

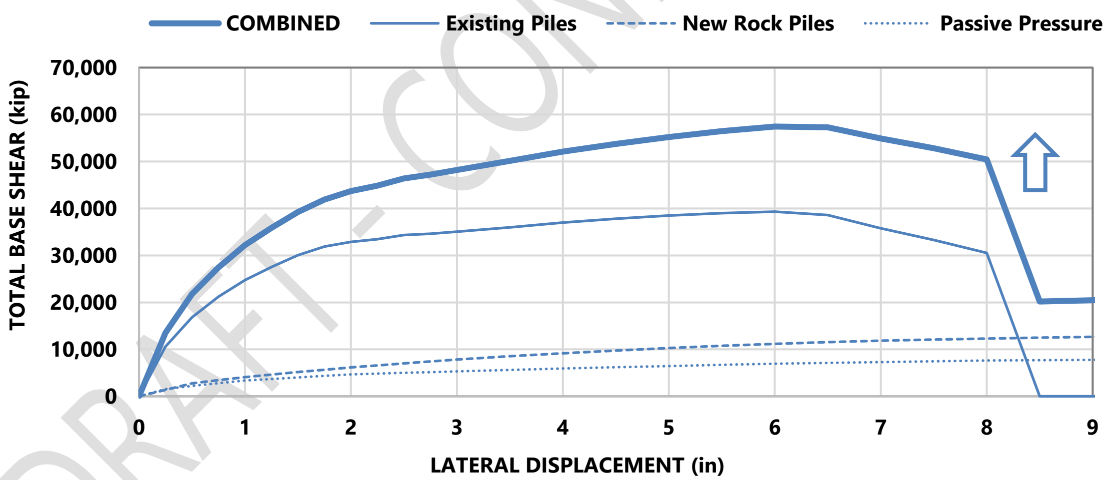


Figure 6-5. Global Foundation Pushover – Pushing North

6.2.2.3. Tower Mat

The tower mat flexural and shear performance will be reviewed per the PEER guidelines as a force-based ordinary element. The impact of the holes cut in the mat for the new retrofit piles will be evaluated throughout construction at the major construction milestones to verify that the calculated demand-capacity ratios are limited to 1.0.

Representative examples of the mat review approach accounting for the holes are included below. Detailed calculations will be provided in the forthcoming calculation packages.

Flexure

The tower mat is divided into design strips for review of its flexural response. The strip locations are determined based on the locations of the primary column elements. Column strip widths are determined per the recommendations of ACI 318-14 §8.11.2. The 11 strips in E-W direction and 7 strips in N-S direction are shown in Figure 6-6.

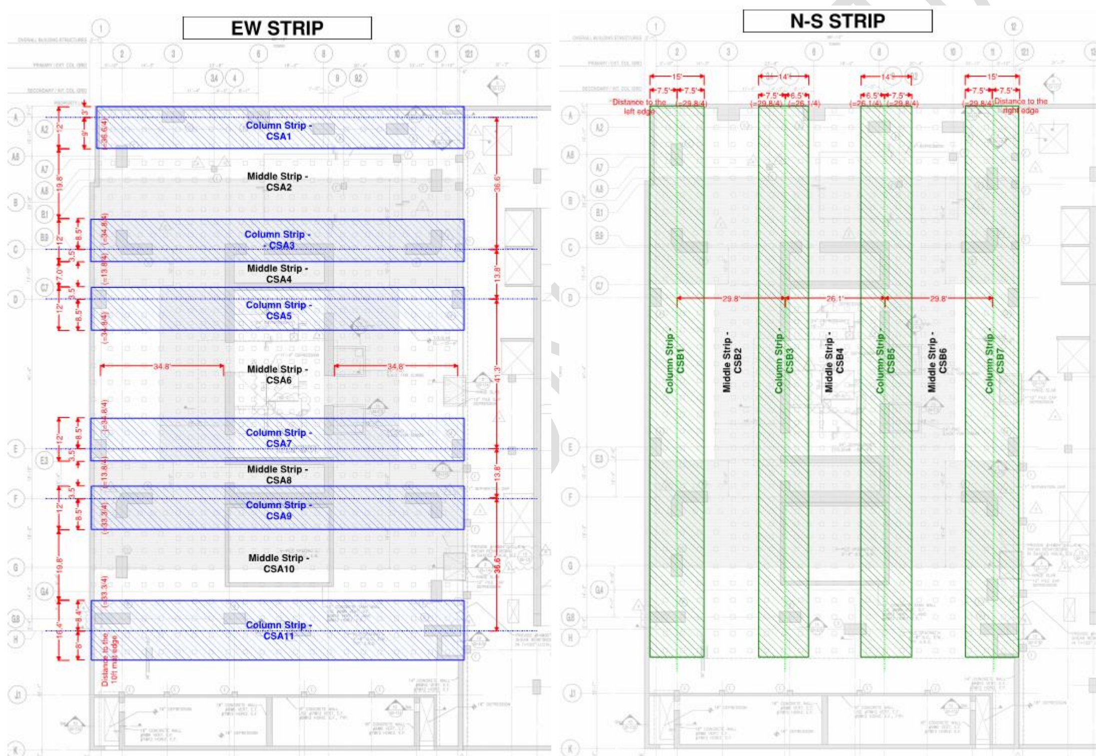


Figure 6-6. Mat Column and Middle Strips

The typical spacing of mat reinforcement is 6” o.c. Each pile will require a 16” diameter core hole. We assume that each 16” hole will cut a maximum of 3 bars in each layer in the strip span direction being considered. Once a bar is cut, it must be developed on each side of the hole. Partial bar development is considered where holes are near points of maximum moment. Note that the reinforcement arrangement has been determined from the original project approved rebar shop drawings. These drawings are provided for reference in Appendix C.2.

The design flexural capacity is calculated following the procedure below:

1) Area of longitudinal reinforcement is calculated along the length of the strip
2) The pile hole locations are overlaid on the design strip
3) The area of steel that will be cut is calculated
4) Development lengths for the cut bars are calculated
5) The design reinforcement is calculated along the length of the strip as:

$$As,eff = As,uncut + As,cut\left(\frac{x}{Ld}\right)$$, where x = distance from core hole (≤ Ld)

An example of calculation is provided in Figure 6-7 for the strip along outrigger line C.

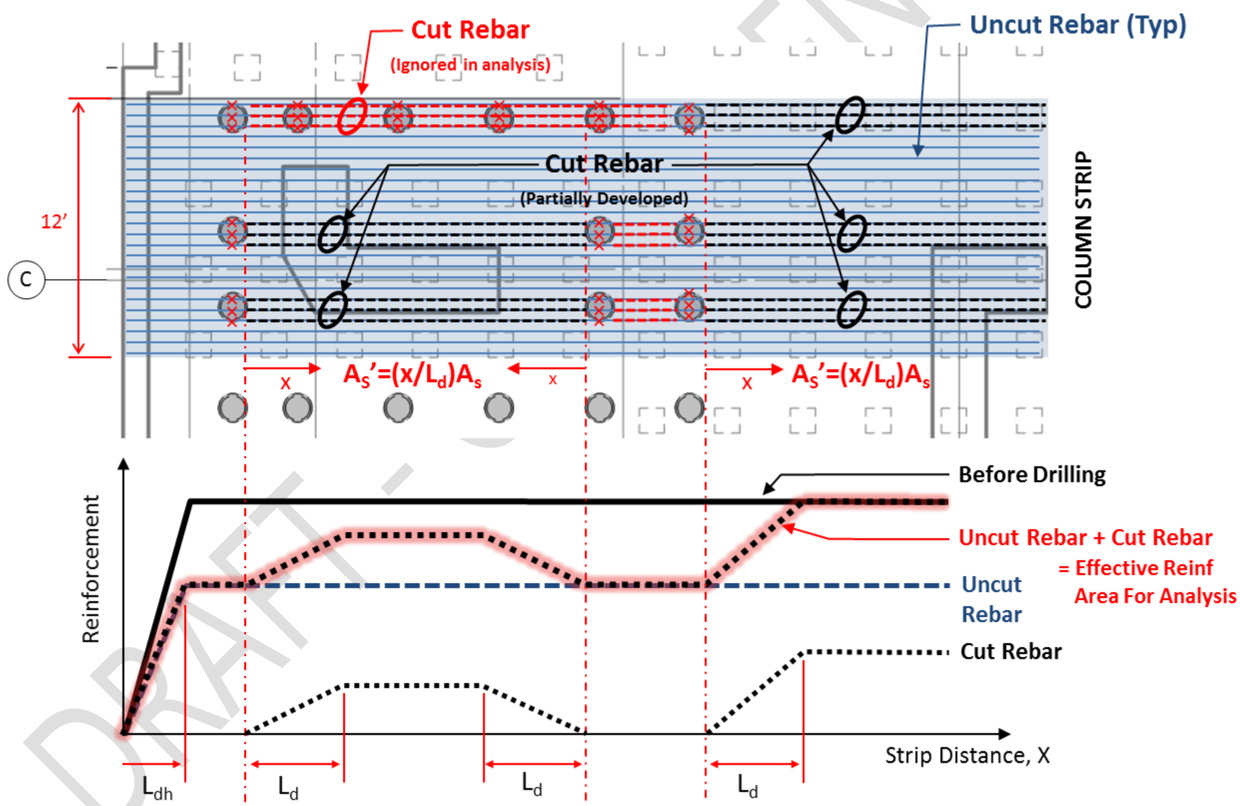


Figure 6-7. Effective flexural reinforcement after core holes for new piles

<u>Shear</u>

The tower mat is also reviewed for its resistance to punching shear considering the core holes required for new pile installation. The punching shear strength is checked at the critical shear section which is recommended by ACI 318-14 to be located at $d/2$ from the face of column with perimeter b_0, where d is the effect depth of the mat slab, as indicated in Figure 6-8.

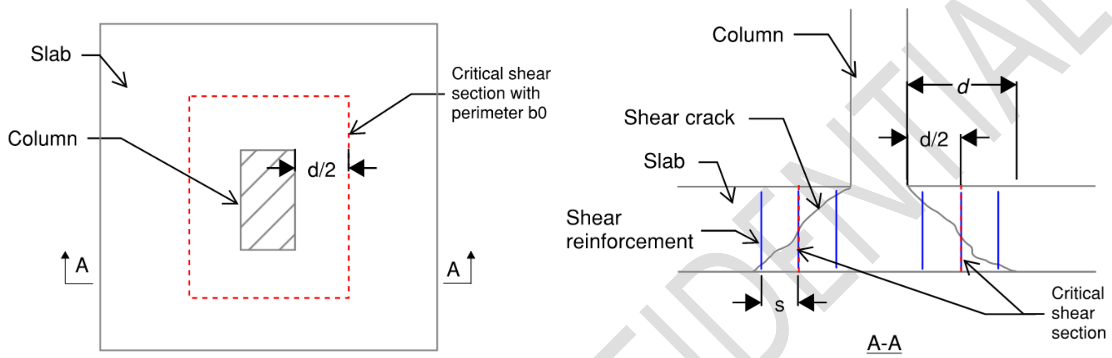


Figure 6-8. Critical Shear Perimeter per ACI 318-14

Where core holes for new piles are located within d of the column face, the critical shear perimeter is reduced. The effective shear perimeter is calculated as illustrated in Figure 6-9.

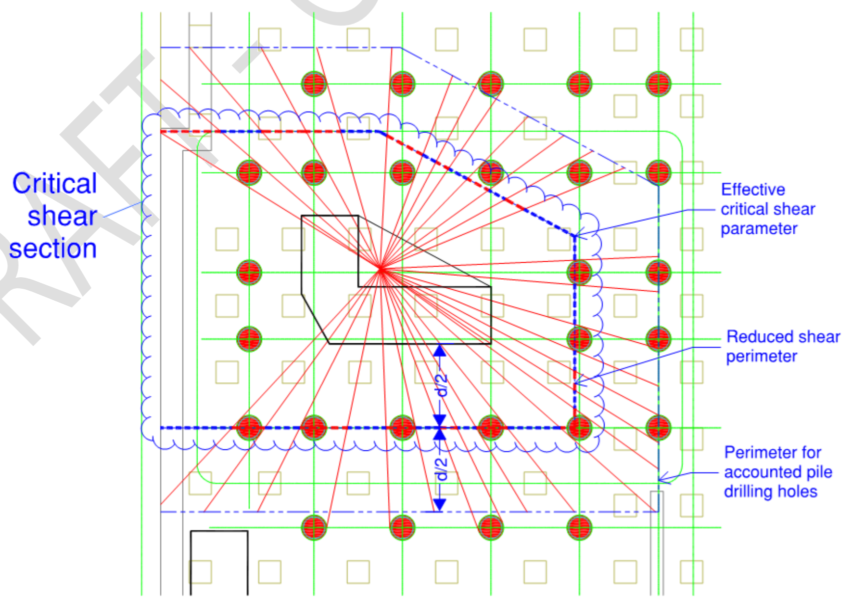


Figure 6-9. Mat Punching Shear Capacity reduction due to core holes for new piles[AJK4]

The punching shear demand, V_u , is calculated as the net of the axial force in the columns and forces in the piles, i.e., $V_u = P_u - P_{pile}$, as indicated in Figure 6-10.

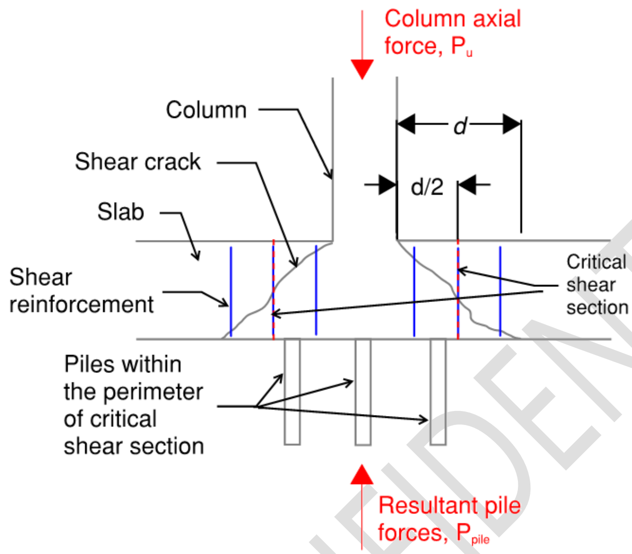


Figure 6-10. Net Punching Shear

Unbalanced moments

Two cases are considered:

1) Punching shear at each individual column (Figure 6-11, left)
2) Punching shear at groups of relatively closely spaced columns (Figure 6-11, right)

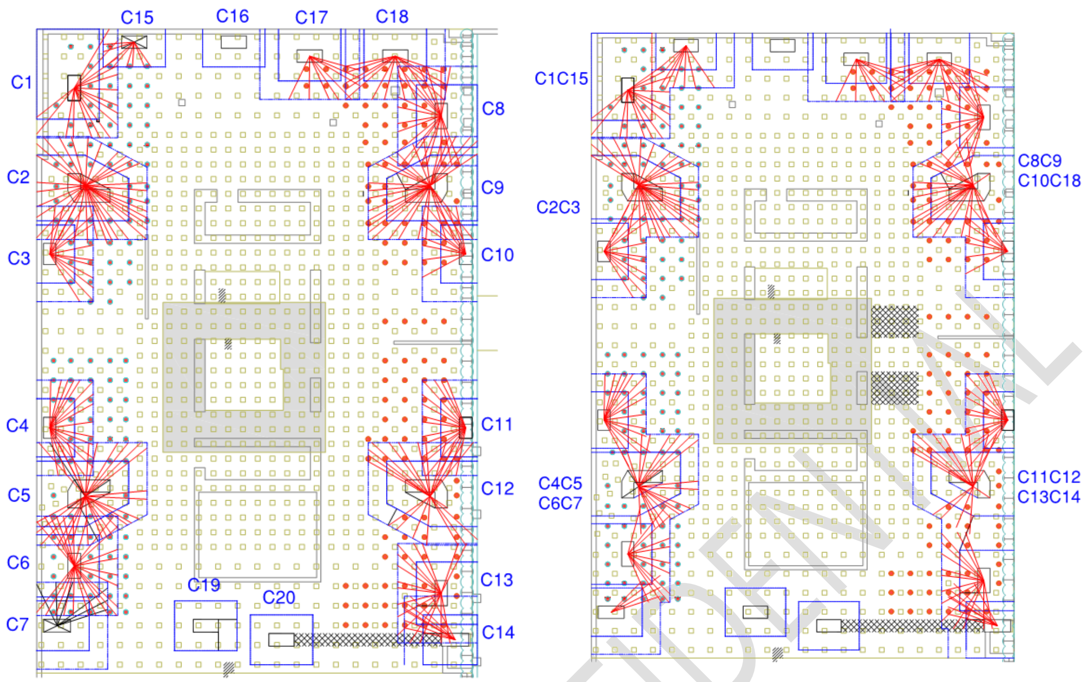


Figure 6-11. Mat Critical Perimeters for Punching Shear[AJK5]

7 ANALYSIS APPROACH

7.1. Analytical Modeling

7.1.1. Analysis Software

The gravity force distribution and settlement analysis is conducted using the staged construction analysis feature of SAP2000 v18. The verification of the foundation stabilization design is conducted using a nonlinear response history analysis in ETABS 2016 v16.2. A screenshot of the ETABS model is shown in Figure 7-1.

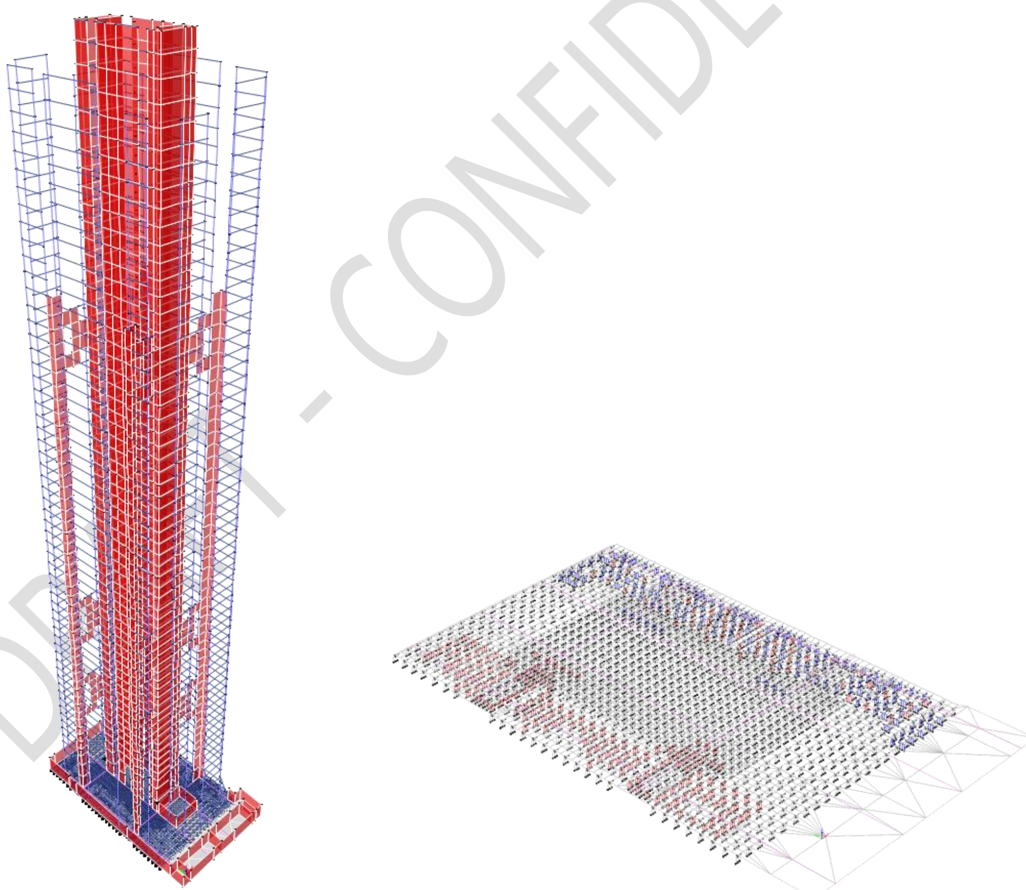


Figure 7-1. ETABS NLRHA Model

7.1.2. Modeling Procedures and Assumptions

Core walls, basement walls, and the L-shaped outrigger columns are modeled as shell elements. Coupling beams, moment frame beams, and moment frame columns are modeled as frame elements. The mat is modeled using shell elements.

The model is "fixed" at the top of the mat foundation. All joints at this level are vertically and horizontally supported (without rotational restraints). Vertical and horizontal springs have been developed with ENGEO, accounting for nonlinearity in both the geotechnical and structural response to load.

The SAP2000 elastic model has floor slabs modeled as shell elements, with superimposed gravity loads applied to the floor slab shell elements. Floor elements have been omitted from the ETABS response history model to reduce computational time. For this model, superimposed gravity loads are calculated according to tributary areas and applied directly to the beams and/or columns that support the slab.

7.1.2.1. Diaphragms

Floor slab diaphragms are modeled as rigid diaphragms.

7.1.2.2. Mass

Mass in accordance with PEER *Guidelines* §4.2.5 has been applied in the models. The self-weight of the shear walls, basement walls, columns, slabs, and mat are calculated internally in the analytical models. All other loading, such as distributed superimposed dead load, cladding load, and live loads, are applied as described in Section 7.1.2.

7.1.2.3. Superstructure Damping

1.5% equivalent viscous damping is used in the SLE analysis per PEER *Guidelines* §4.2.7, while 2.5% equivalent viscous damping is used for DBE and MCE analysis.

7.1.2.4. Foundation Damping

Damping at the soil-foundation interface is neglected. Per PEER *Guidelines* §4.5.2 and trial analyses performed by LERA, this is a conservative simplification as the potentially beneficial effects of radiation damping are not included in the model.

7.2. Bounding for Potential Geotechnical Uncertainties

Load is transferred from existing foundation elements to new piles through a combination of initial preload and continued settlement of the tower. It is assumed that as long as settlement is still occurring, load transfer will continue. Further, it is assumed that as the existing piles are unloaded (and the Old Bay Clay is thus unloaded) the consolidation settlement rate (and thus load transfer rates), will slow.

Since the performance of the retrofit design is in large part driven by the assumptions made regarding relative stiffness between existing and new foundation elements, it is important that the solution be bounded to account for potential uncertainties in the geotechnical properties.

In the absence of more sophisticated analysis, ASCE 41-13 recommends a prescriptive approach to geotechnical bounding, using a wide range of potential capacities from Q/2 to 2Q, where Q is the calculated capacity. See Figure 7-2. Based on the extensive data accumulated at this site, ENGEO has been able to provide a narrower bounding than the ASCE 41 approach. Upper and lower bound component properties for each element type will be discussed in detail in the following sections.

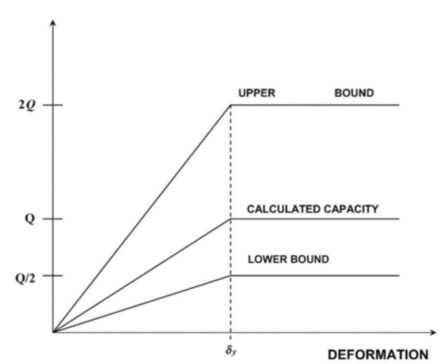


Figure 7-2. Idealized Elastoplastic Load-Deformation from Behavior for Soils (from ASCE 41-13 Fig 8-1(a))

7.3. Gravity & Foundation Settlement Analysis

The gravity force distribution and settlement analysis is conducted using the staged construction analysis feature of SAP2000 v18. The stages of the analysis are summarized below:

1) Start of construction to today
    a. Build model
    b. Support springs at each existing pile with effective stiffness representing the long term settlement behavior of the foundation (Figure 7-4 and Figure 7-5)
    c. Apply gravity loads (1.0D + $L_{Expected}$, where $L_{Expected}$ = 0.5$L_{Reduced}$)
    d. The deflected position of the mat is checked against the surveyed position (Figure 7-6 and Figure 7-7)
2) Install new piles
    a. It is assumed that piles are installed, preloaded, and locked-off in groups of approximately 5 piles at a time to capture expected losses associated with adjacent preloading operations
    b. Existing piles have dynamic (short-term) stiffness properties (Figure 7-4)
    c. New piles have dynamic (short-term) stiffness properties
    d. Preloads to the new pile elements are applied by internal temperature loads
3) Post-Retrofit
    a. ENGEO has provided upper and lower bound estimates of expected future settlement once the full retrofit is installed
    b. Existing piles have static (long-term) stiffnesses calibrated to allow the final increment of settlement predicted by ENGEO, until settlement finally arrests.
    c. New piles have static (long-term) stiffnesses

Three interim milestone moments in time have been identified as critical to check the performance of the retrofit in seismic events. These interim milestones (I-III) have been identified on the staged analysis timeline shown in Figure 7-3. Pile and mat foundation initial conditions (force and deflection) established from the gravity load distribution and foundation settlement analysis model are imposed as the initial conditions in the ETABS NLTHA model for each of the critical milestones.

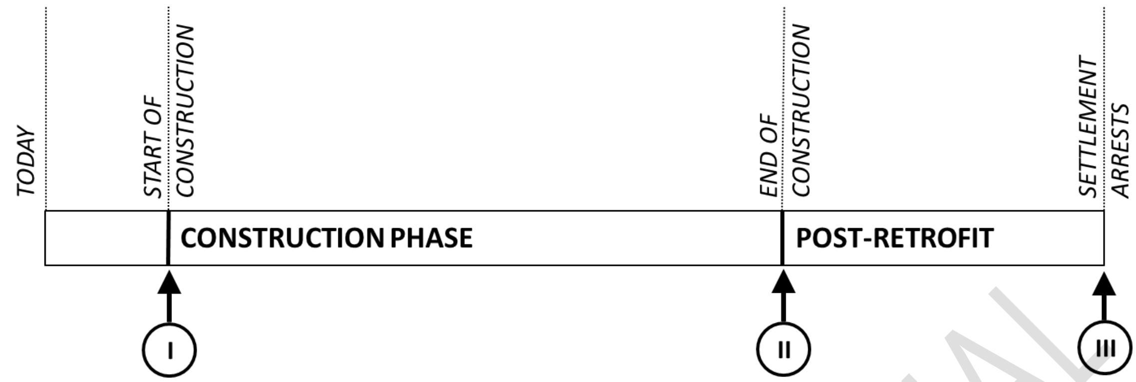


Figure 7-3. Staged Analysis Timeline

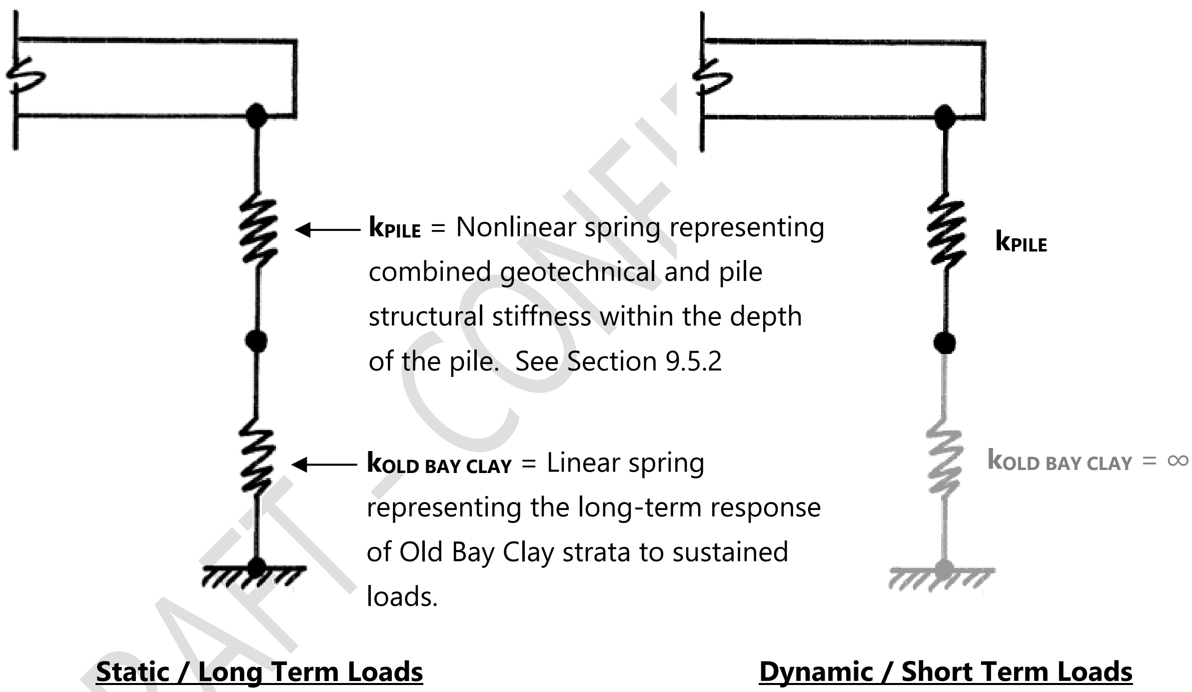


Figure 7-4. Existing Pile Spring Series

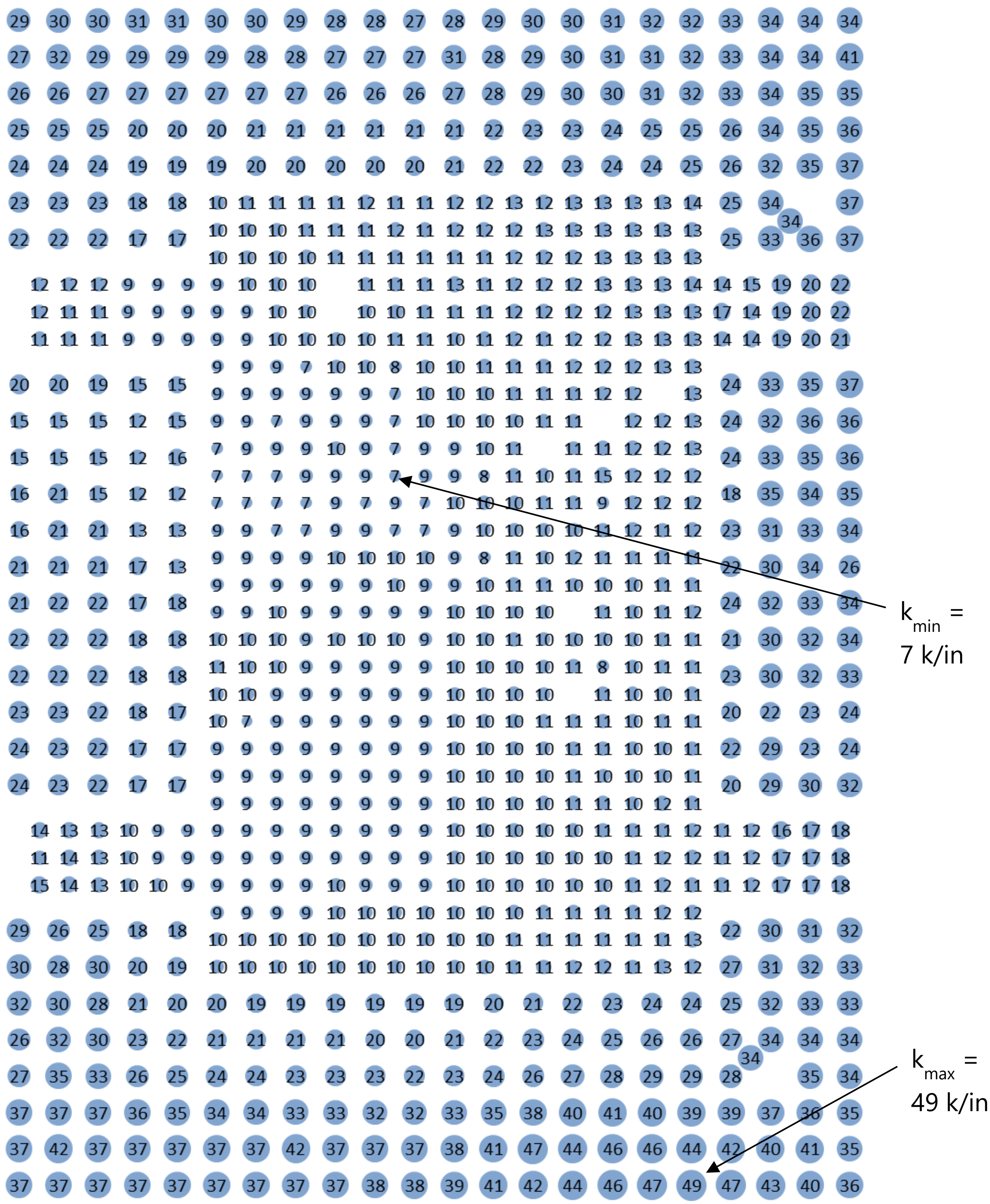


Figure 7-5. Initial Pile OBC Spring Stiffnesses, k/in (Start of Construction to Today)

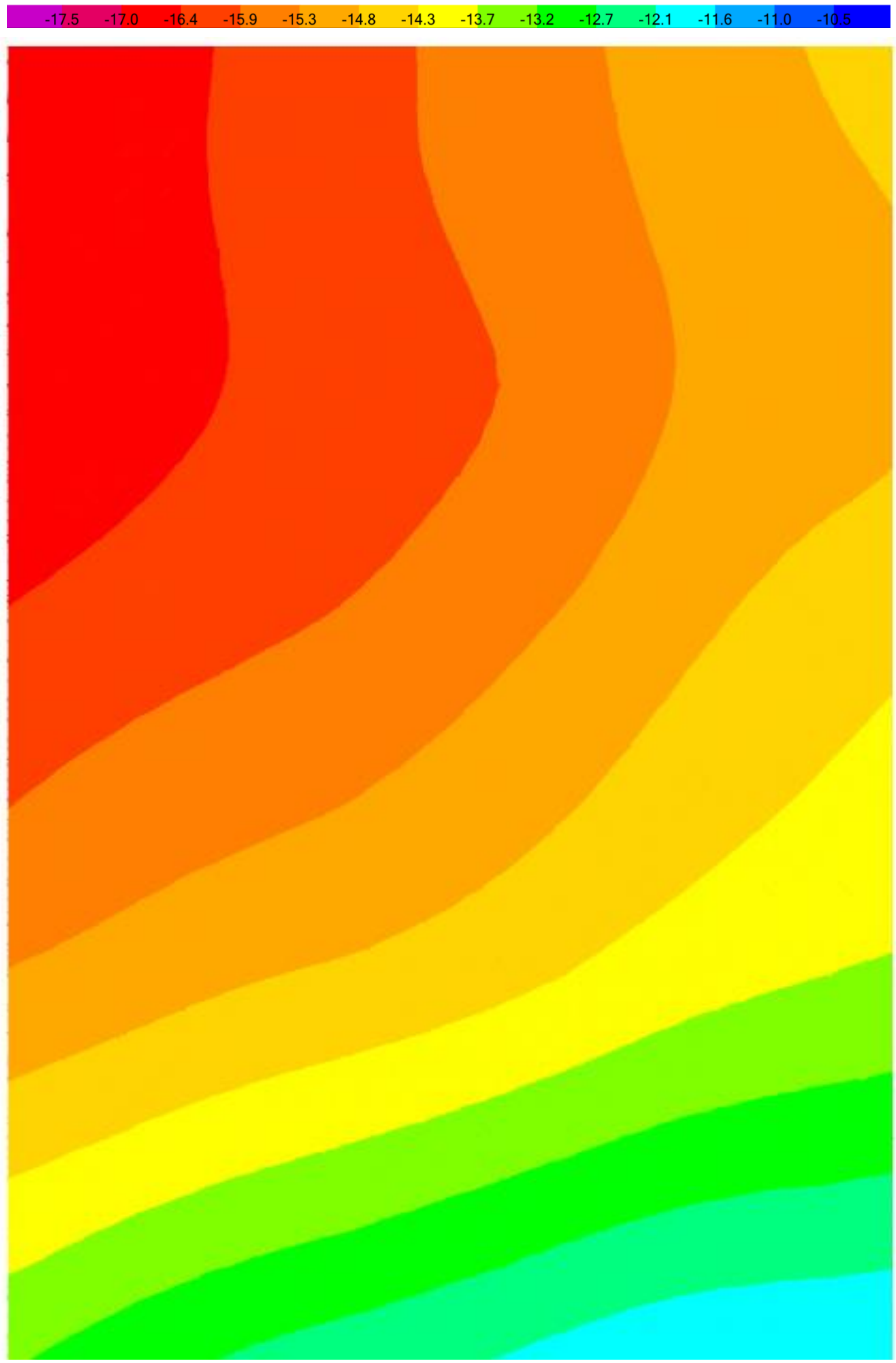


Figure 7-6. Analytical Mat Deflected Shape D + $L_{Expected}$, kips (Today)

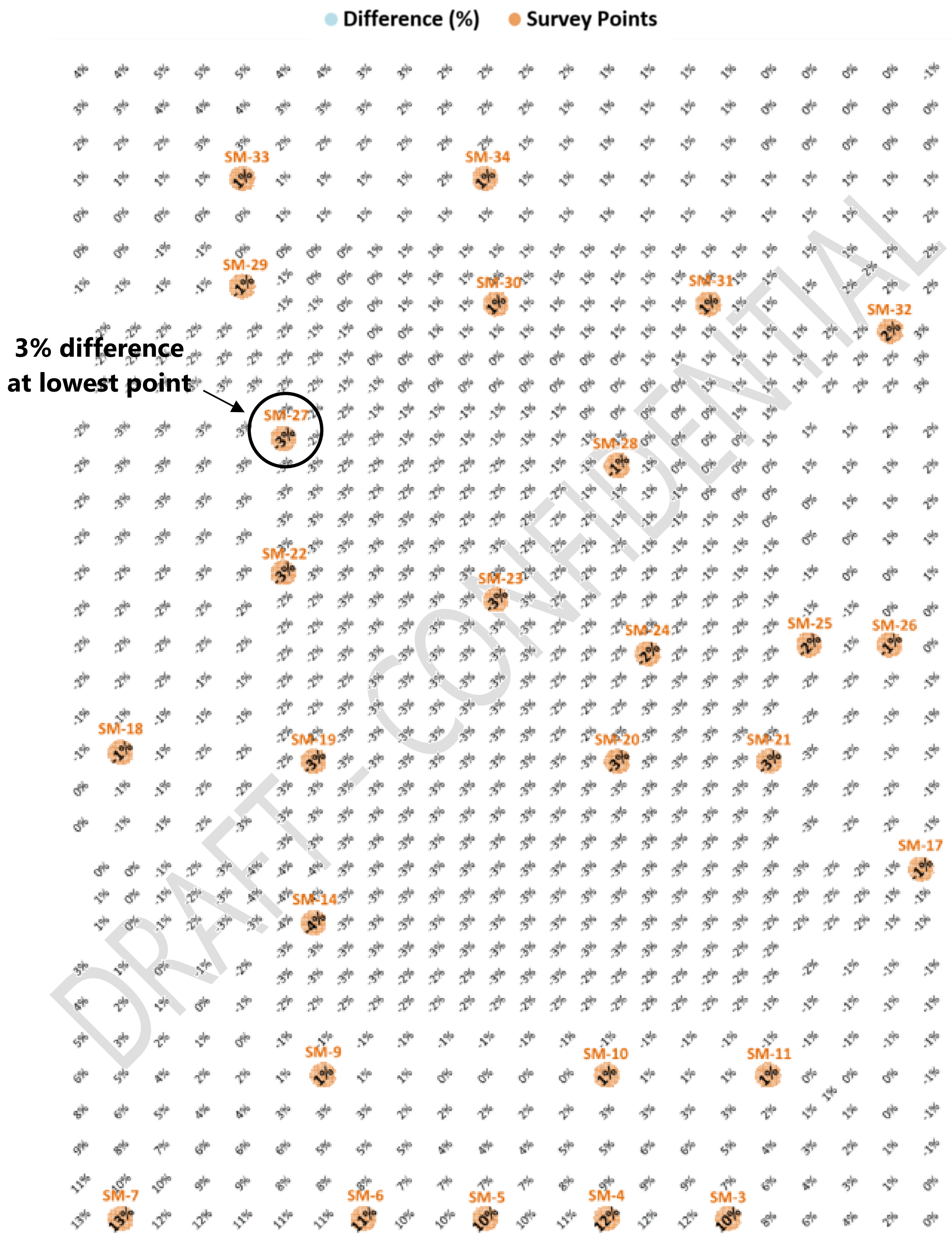


Figure 7-7. Analytical Mat Deflected Shape Compared with Survey Data[AJK6]

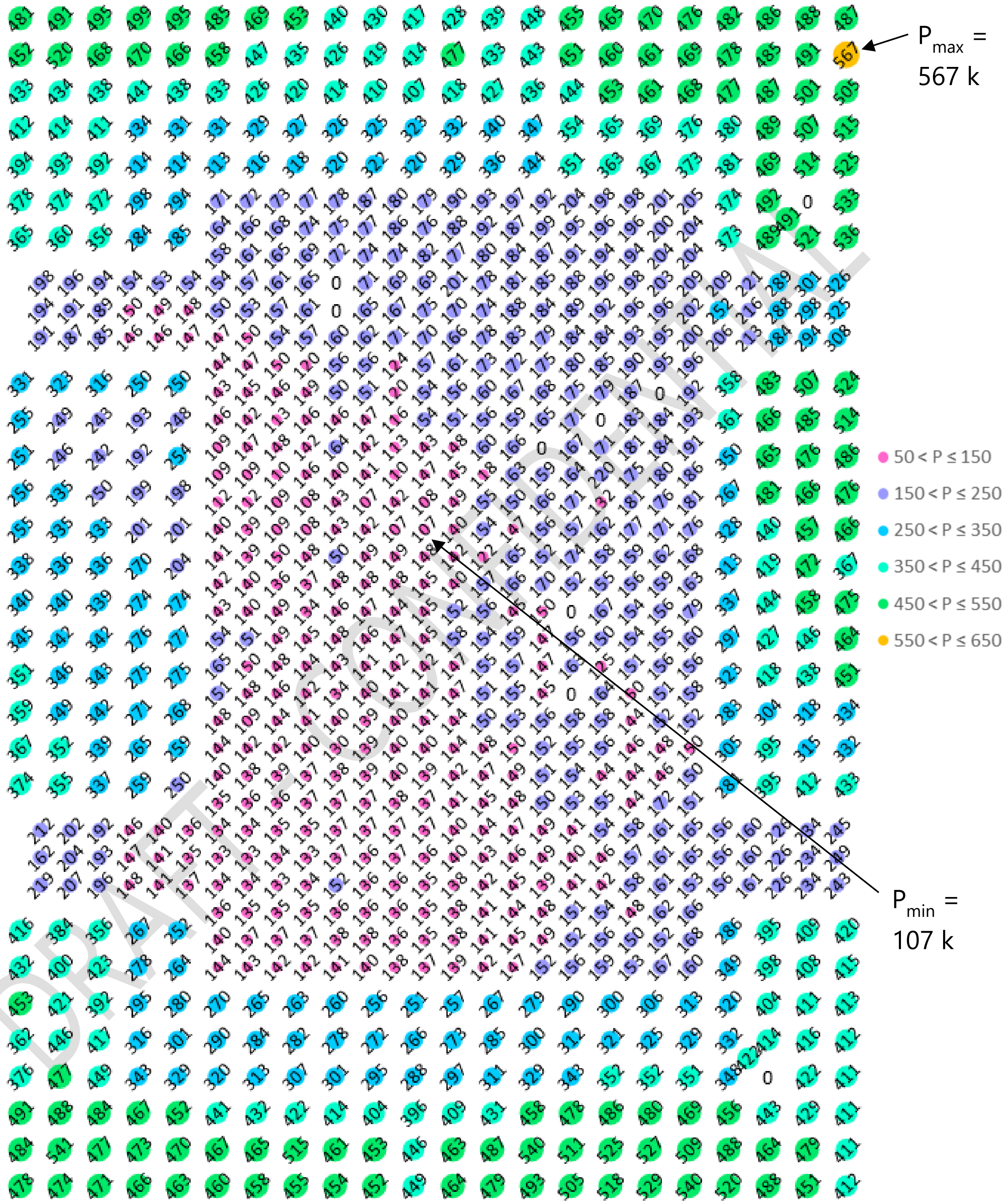


Figure 7-8. Pile Gravity Loads, D + $L_{Expected}$, kips (Today)

<u>**Geotechnical Bounding**</u>

Bounding is used to capture upper and lower bound estimates of the load transfer between the existing and new foundation elements. Bounding is considered for the following events in the retrofit sequence. Also see Figure 7-9.

1) Expected losses due to pile installation sequence
2) Load transfer from existing to new piles during retrofit installation
3) Amount of post-retrofit settlement
4) Relative dynamic stiffness between existing and new foundation elements during seismic events (Figure 7-10)

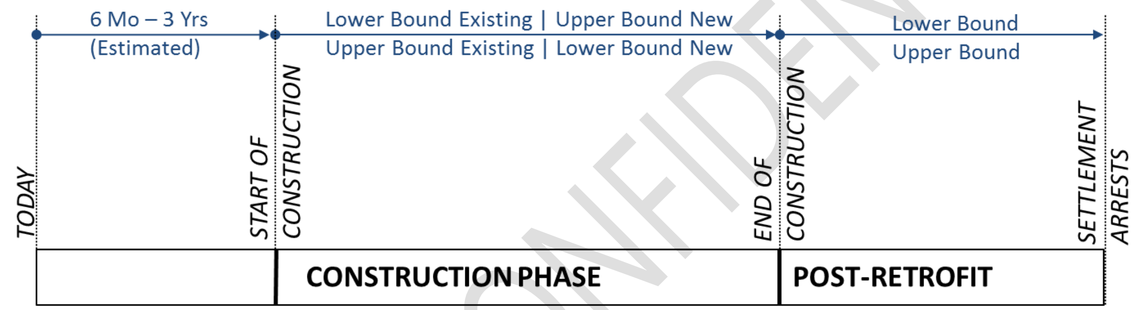


Figure 7-9. Retrofit Events to Bound

Existing Piles	*Dynamic*	UB	UB	LB	LB	AVG
Rock Piles	*Dynamic*	UB	LB	UB	LB	AVG

Figure 7-10. Potential Combinations of Relative Dynamic Stiffness

Considering each of these events individually, there would be many thousands of potential combinations of upper bound, lower bound, and expected properties to bound. We have made the following simplifying assumptions to reduce the number of required analyses. Note that these assumptions are all generally based on the idea that piles reacting against the same soil strata should behave the same way:

1) *For both pile types, the vertical compression and tension capacity-deflection bounding curves should match.* So, if we are analyzing the existing piles using the lower bound compression capacity-deflection curve then it is appropriate to always pair it with the lower bound tension capacity-deflection curve. Similarly, if we are analyzing the upper bound compression curve we always pair it with the upper bound tension curve.

2) *For the Rock Piles, the vertical static & dynamic capacity-deflection bounding curves should match.* So, if we are analyzing the Rock Piles using the lower bound static curve then it is appropriate to always pair it with the lower bound dynamic curve. And if we are analyzing the upper bound static curve then we should always pair it with the upper bound dynamic curve. Note: For the Existing Piles it is assumed the static and dynamic stiffness don't need to match (lower bound static can be paired with upper bound dynamic for example) since the static stiffness is dependent on the OBC and the dynamic stiffness isn't.

4) *For the Existing Piles, the vertical capacity-deflection bounding curve should match between zones.* So, for example, If we are using lower bound curve for the Existing Piles in Zone 1 then we should also use the lower bound curves for the Existing Piles in other zones.

5) *For all pile types, the lateral load-deflection bounding curves should match.* So, for example, if we are using the lower bound lateral stiffness curves for the Rock Piles it is appropriate to use the lower bound lateral stiffness curves for the Existing Piles.

6) *The vertical response of the OBC/Rock and Sand layers are not linked.* So, for example, the existing piles could see upper bound vertical stiffness while the rock piles have lower bound vertical stiffness (and vice versa).

7) *The lateral and vertical response of the piles are not linked (for both pile types).* So, for example, the lateral load-deflection could be lower bound while the vertical load-deflection is upper bound.

Summary of Analyses:

Using these simplifying assumptions and considering only the critical scenario for each foundation component type, we have reduced the number of bounding models down to three. For each of the three bounding models, seismic and wind loads will be applied at the critical milestone moments in the retrofit installation sequence as described in Figure 7-11, Figure 7-12, and Figure 7-13. In all, seismic and wind loads will be evaluated for 6 distinct models.

- Analysis 1: Proportioning Model (Average Stiffness Properties for All Elements)
- Analysis 2: Least Force Transfer: Critical Case for Existing Piles
- Analysis 3: Greatest Force Transfer: Critical Case for Rock Piles

	Vertical Stiffness		Lateral Stiffness
	Static	*Dynamic*	
Existing Piles	Average	Average*	Average
Rock Piles	Average	Average*	Average

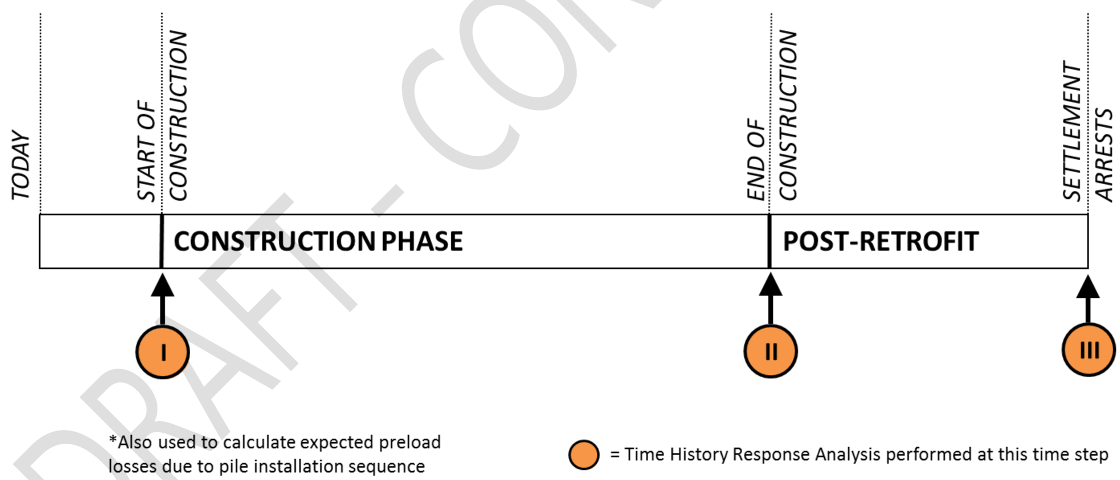
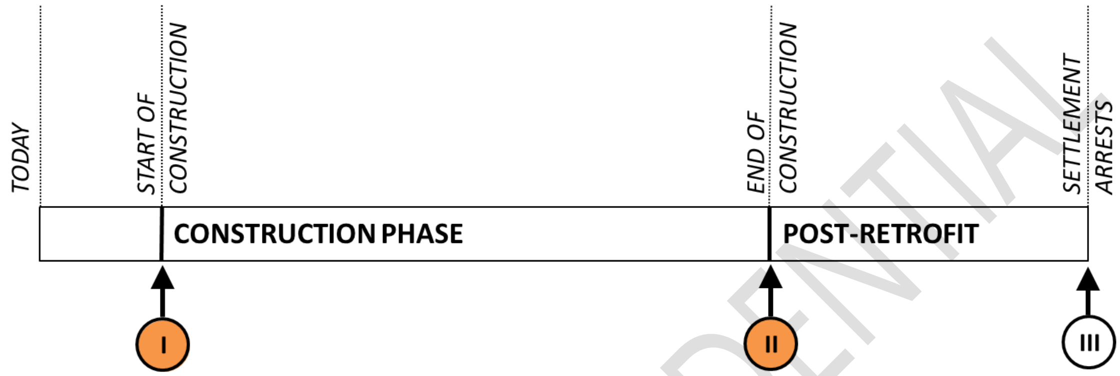


Figure 7-11. Analysis 1: Proportioning Model (Avg Stiffness Properties for All Elements)

	Vertical Stiffness		Lateral Stiffness
	Static	*Dynamic*	
Existing Piles	Upper Bound	Upper Bound*	Lower Bound
Rock Piles	Lower Bound	Lower Bound*	Lower Bound

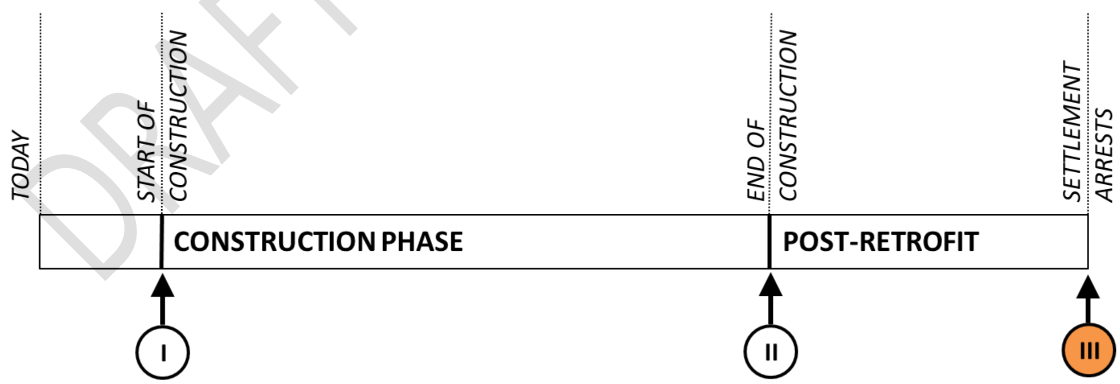


*Also used to calculate expected preload losses due to pile installation sequence

= Time History Response Analysis performed at this time step

Figure 7-12. Analysis 2: Least Force Transfer: Critical Case for Existing Piles

	Vertical Stiffness		Lateral Stiffness
	Static	*Dynamic*	
Existing Piles	Lower Bound	Lower Bound*	Lower Bound
Rock Piles	Upper Bound	Upper Bound*	Lower Bound



*Also used to calculate expected preload losses due to pile installation sequence

= Time History Response Analysis performed at this time step

Figure 7-13. Analysis 3: Greatest Force Transfer: Critical Case for Rock Piles

7.4. Analysis for Wind and DBE

The initial design of the lateral force-resisting system is conducted using a SAP2000 v18 elastic analysis model. The Code-mandated DBE forces are determined using the Modal Response Spectrum procedure. The Code-mandated wind forces are determined using the Directional procedure.

Pile and mat foundation initial conditions are established from the gravity load distribution and foundation settlement analysis. The wind and DBE level forces are applied at the 6 critical construction stages shown in Section 7.2 to verify performance through the entirety of the retrofit installation.

7.4.1. Component Properties

7.4.1.1. Superstructure Component Properties

The effective stiffnesses of structural components in the elastic analysis model are listed in Table 7-1. These stiffnesses are adopted from ACI 318-14 Table 6.6.3.1.1(a). Where ACI 318-14 does not provide guidance on the effective stiffness of a type of component, the effective stiffness is based on PEER *Guidelines* Table 4-3.

Table 7-1. Effective stiffnesses for the Wind and DBE model

Component	Axial stiffness	Flexural stiffness	Shear stiffness
Reinforced concrete structural walls	– Note 1	$0.7E_cI_g$	G_cA_g
Reinforced basement walls	– Note 1	$0.7E_cI_g$	G_cA_g
Coupling beams			
Composite	E_cA_g	$0.06(\ell/h)E_cI_g$	G_cA_g
Conventionally reinforced	E_cA_g	$0.07(\ell/h)E_cI_g \le 0.3E_cI_g$	G_cA_g
Reinforced concrete columns	E_cA_g	$0.7E_cI_g$	G_cA_g
Reinforced concrete beams	E_cA_g	$0.35E_cI_g$	G_cA_g
Post-Tensioned concrete slabs Note 2	– Note 1	$0.5E_cI_g$	$0.5G_cA_g$
Reinforced concrete slabs Note 2	– Note 1	$0.25E_cI_g$	$0.25G_cA_g$
Mat (in-plane)	$0.5E_cA_g$	$0.5E_cI_g$	$1.2G_cA_g$
Mat (out-of-plane)	–	$0.5E_cI_g$	G_cA_g

1. Since these members are modeled with shell elements, the in-plane axial stiffness modifier is identical to the in-plane flexural stiffness modifier.
2. Slabs are modeled with a sufficiently low stiffness modifier such that the tower seismic force resisting system receives essentially all of the Wind and DBE forces.

7.4.1.2. Foundation Component Properties

The model is connected to springs at the piles. Each existing and new pile is represented by a vertical spring and pair of horizontal springs. These spring stiffnesses are the same as used for MCE_R analysis, see Section 7.6.2.

7.5. Analysis for SLE

The verification of the foundation stabilization design for the Service Level Earthquake is conducted using a nonlinear response history analysis using ETABS 2016 v16.2.

Pile and mat foundation initial conditions are established from the gravity load distribution and foundation settlement analysis. The Service level ground motions are applied at the 6 critical construction stages shown in Section 7.2 to verify performance through the entirety of the retrofit installation.

7.5.1. Component Properties

7.5.1.1. Superstructure Component Properties

Although limited to no yielding is expected in a Service Level event, components are modeled with the same nonlinear properties as in the MCE model. See Sections 7.6.1.1 through 7.6.1.4.

7.5.1.2. Foundation Component Properties

The model is connected to springs at the piles. Each existing and new pile is represented by a vertical spring and pair of horizontal springs. These spring stiffnesses are the same as used for MCE_R analysis, see Section 7.6.2.

7.6. Analysis for MCE

The verification of the foundation stabilization design for the MCE is conducted using a nonlinear response history analysis using ETABS 2016 v16.2.

Pile and mat foundation initial conditions are established from the gravity load distribution and foundation settlement analysis. The MCE level ground motions are applied at the 6 critical construction stages shown in Section 7.2 to verify performance through the entirety of the retrofit installation.

7.6.1. Superstructure Component Properties

The effective pre-yield stiffnesses of structural components in the analysis model are listed in Table 7-1. These stiffnesses are based on PEER *Guidelines* Table 4-3, except where there are specific recommendations in literature.

Table 7-2. Effective stiffnesses for the MCE model

Component	Axial stiffness	Flexural stiffness	Shear stiffness
Reinforced concrete structural walls	– Note 1	– Note 1	$0.5G_cA_g$
Reinforced basement walls	–	$0.8E_cI_g$	$0.5G_cA_g$
Coupling beams			
Composite	E_cA_g	$0.06(\ell/h)E_cI_g$	G_cA_g
Conventionally reinforced	E_cA_g	$0.07(\ell/h)E_cI_g \leq 0.3E_cI_g$	G_cA_g
Reinforced concrete beams	E_cA_g	$0.3E_cI_g$	G_cA_g
Reinforced concrete columns	E_cA_g	$0.7E_cI_g$	G_cA_g
Diagonally-reinforced outriggers	E_cA_g	$0.5E_cI_g$	$0.5G_cA_g$
Mat (in-plane)	$0.5E_cA_g$	$0.5E_cI_g$	$1.2G_cA_g$
Mat (out-of-plane)	–	$0.5E_cI_g$	G_cA_g

1. See Section 7.6.1.1.

7.6.1.1. Reinforced Concrete Structural Walls

Core walls are modeled in axial/flexure using fiber elements, where fibers represent the concrete and reinforcing steel in the walls. The material properties for each are discussed below.

Since the core walls have been designed using prescriptive code methods, they are expected to yield in shear under MCE. The shear model for core walls is described below.

Concrete:

Concrete properties are modeled with confined and unconfined concrete stress-strain relationships from Mander et al. (1988) based on the level of confinement provided in the Construction Documents. The material model has no stiffness or strength in tension. Figure 7-14 shows a sample stress-strain curve for confined concrete.

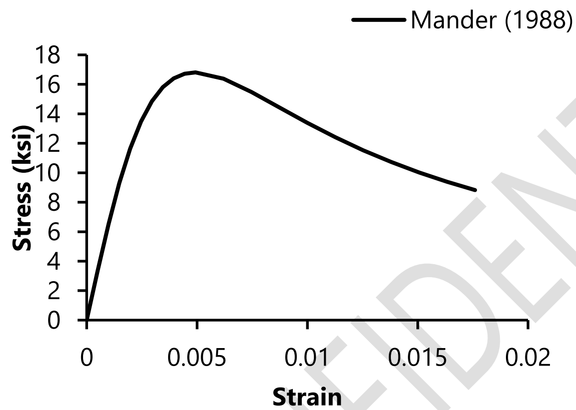


Figure 7-14. Sample Mander (1988) concrete model

Reinforcing Steel:

Rebar is modeled with the stress-strain relationship shown in Figure 7-15 and Figure 7-16 for both tension and compression. The relationship is developed based on the material properties in

Table 13-3 and elongation requirements of ASTM A615 and ASTM A706.

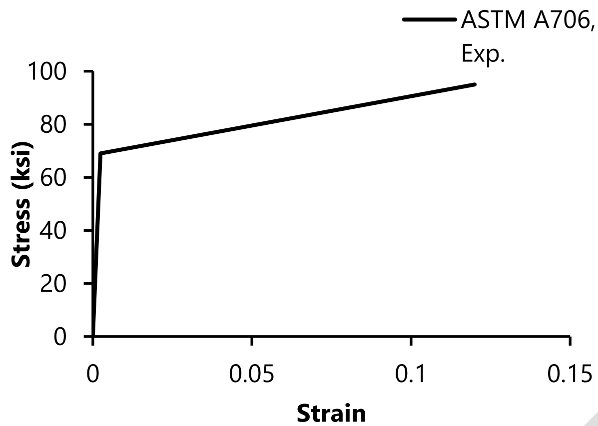


Figure 7-15. ASTM A706 Grade 60 reinforcing steel model

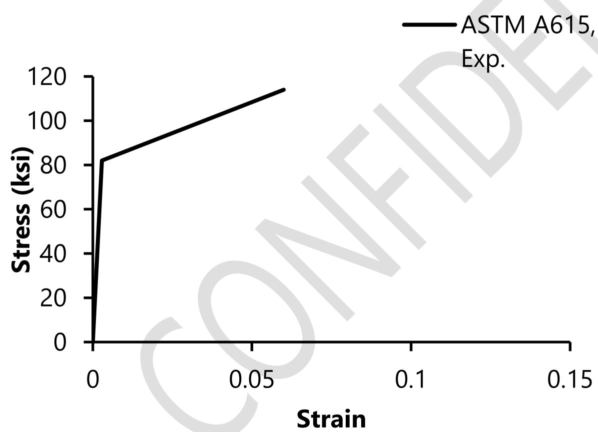


Figure 7-16. ASTM A615 Grade 75 reinforcing steel model

Shear:

Since the existing structure was designed with Code-based rules that do not require overstrength for wall shear design, the existing shear walls may yield in shear in an MCE event. As the longitudinal strain demands are expected to be moderate, the expected shear yield strength is calculated per PEER *Guidelines* Equation 4-2:

$$V_{ne} = 1.5A_{cv}(2\lambda\sqrt{f'_{ce}} + \rho_t f_{ye}) \leq 15A_{cv}\sqrt{f'_{ce}}$$

A_{cv} = Wall shear area
λ = lightweight concrete coefficient
f'_{ce} = expected concrete strength
ρ_t = ratio of transverse reinforcement

f_{ye} = expected transverse reinforcement yield strength

7.6.1.2. Coupling Beams and Moment Frame Beams

Coupling beams and moment frame beams are modeled using the plastic hinge model shown in Figure 7-17 and the parameters in ASCE 41-13, except for composite coupling beams. Composite coupling beams are modeled using parameters calibrated to testing by Motter et al[AJK7]. The yield strength of each composite coupling beam section is determined with Structure Point spColumn v4.81. A sample yield surface and a sample moment-rotation relationship for a composite coupling beam is shown in Figure 7-17 and Figure 7-18.

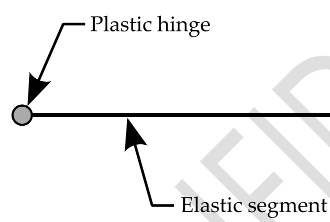


Figure 7-17. Plastic hinge model

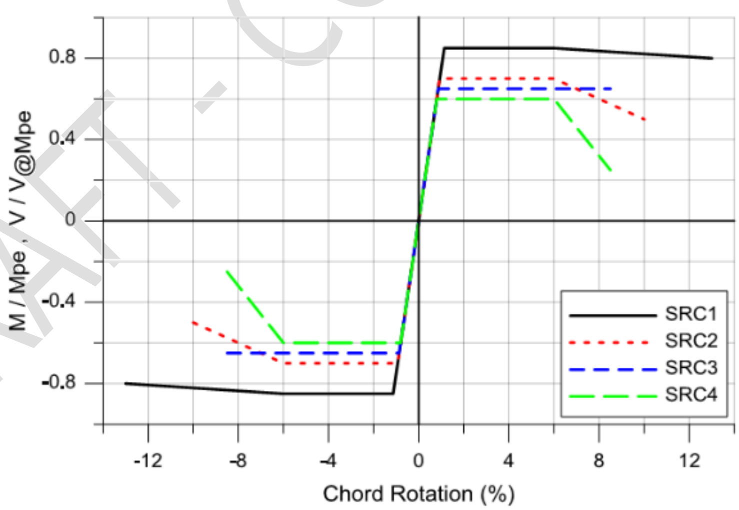


Figure 7-18. Sample moment-rotation relationship

7.6.1.3. Moment Frame Columns

Moment frame columns are modeled with the plastic hinge model shown in Figure 7-17 and the parameters in ASCE 41-13. The hinges consider axial-moment interaction using the yield surface shown in Figure 7-19, and the yield strengths for each column section is determined with Structure Point spColumn v4.81. A sample yield surface and a sample moment-rotation relationship are shown in Figure 7-20 and Figure 7-21.

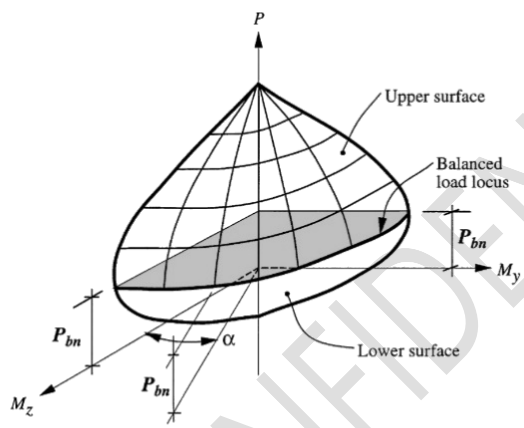


Figure 7-19. Generalized yield surface

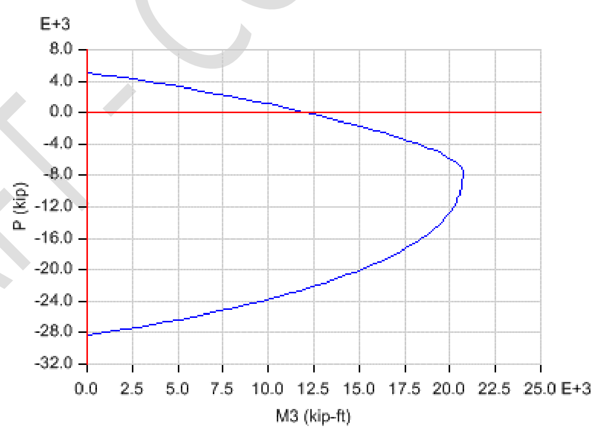


Figure 7-20. Sample yield surface

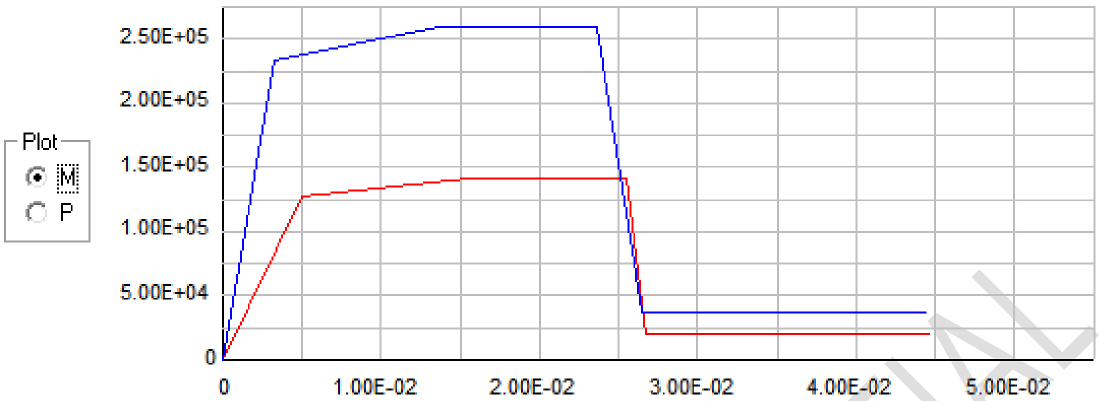


Figure 7-21. Sample moment-rotation relationship

7.6.1.4. Diagonally-Reinforced Outriggers

The diagonally-reinforced outriggers at floors 8, 12, 17, 21, 42 and 46 are modeled with nonlinear shear panels with a backbone curve as shown in Figure 7-22. The backbone curve is based on research on diagonally reinforced beams with low shear span to depth ratios, such as Canbolat et al. (2005) and Galano and Vignoli (2000). Since specimen 1 in Canbolat et al. was not loaded to failure, the ultimate drift is based on ASCE 41-17 and Galano and Vignoli.[AJK8]

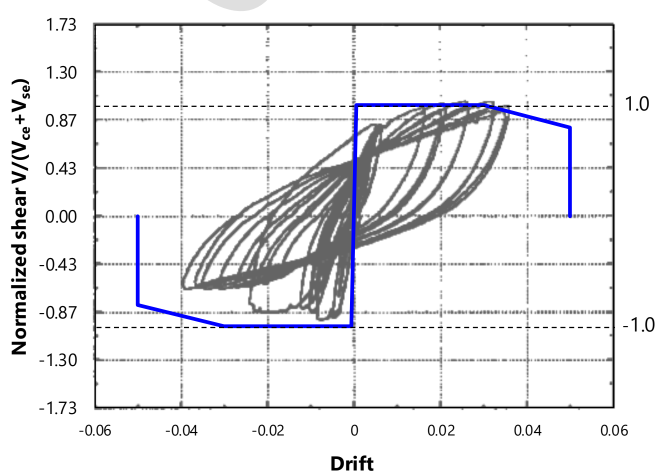


Figure 7-22. Diagonally-reinforced outrigger model (Canbolat et al. specimen 1 data in background)

7.6.2. Foundation Component Properties

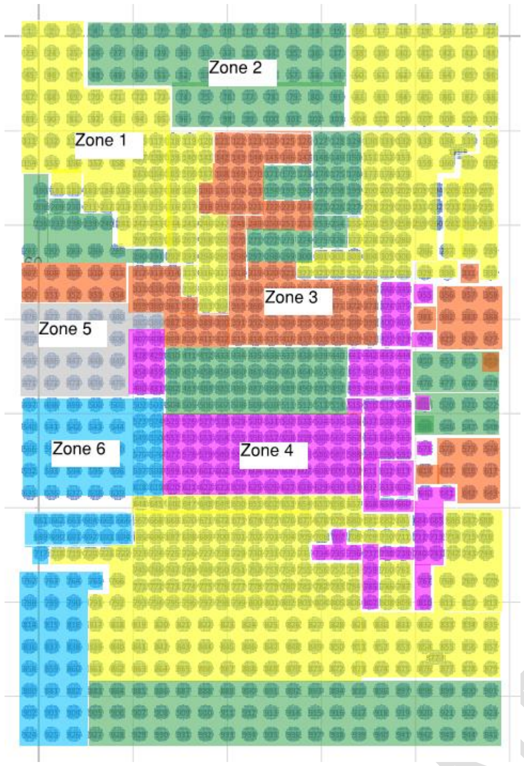
7.6.2.1. Vertical Pile Stiffness

The subsurface conditions vary across the site. ENGEO has provided a mapping of capacity and stiffness of the existing piles by zone. The mapping of zones for existing piles is shown in Figure 7-23.

The stiffness values provided by ENGEO consider the combined effect of pile elastic shortening and geotechnical resistance (end bearing and skin friction). The resulting nonlinearity represents the yielding of the soil under load. Note that these curves plateau at values less than the structural capacity for all pile types. [AJK9]The backbone stiffness curves (using best estimate soil properties) for existing piles are shown in Figure 7-24.

Upper and lower bound estimates of pile capacity and stiffness have been provided by ENGEO. The bounding curves for existing piles are shown in Figure 7-25.

Stiffness and capacity of new rock piles are not a function of position as the bedrock composition does not vary significantly across the site. They do, however, respond differently under short-term and long-term loads. Backbone stiffnesses for the rock piles, including upper and lower bound estimates, for dynamic (short-term) and static (long-term) loads are provided in Figure 7-26 and Figure 7-27.



ZONE	Ultimate Capacity (Expected)	Allowable Capacity (Min F.S. = 2.0)
1	775 k	388 k
2	667 k	334 k
3	505 k	253 k
4	819 k	410 k
5	567 k	284 k
6	707 k	354 k

Figure 7-23. Existing Pile Capacity Zones

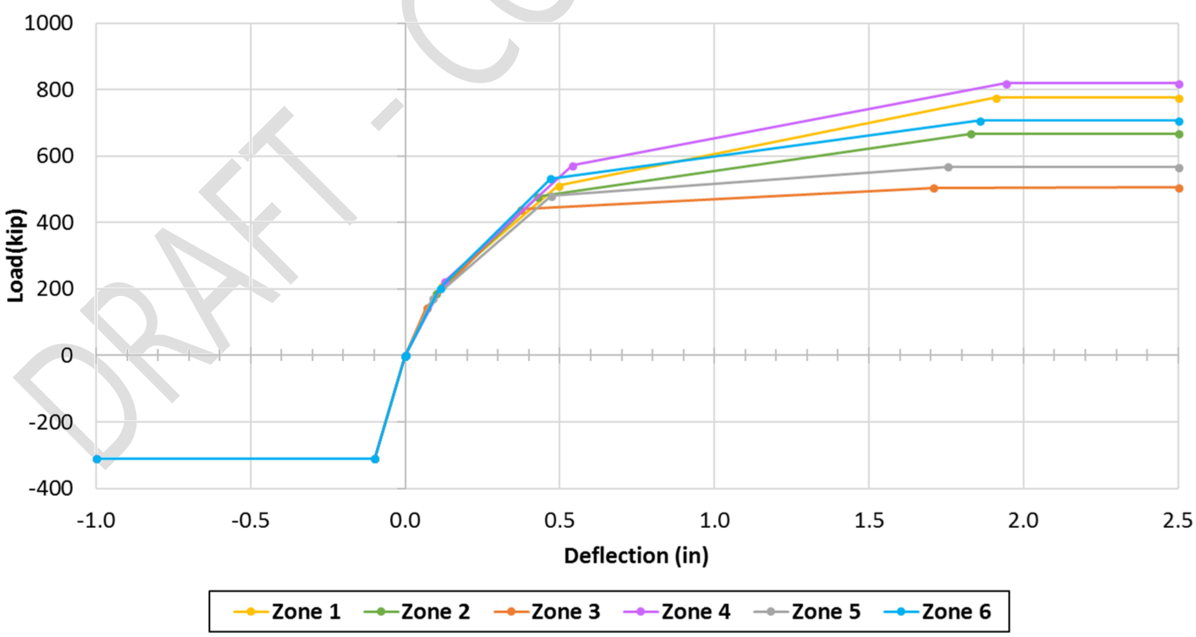


Figure 7-24. Existing Pile Backbone Curves by Zone (Best Estimate Soil Properties)

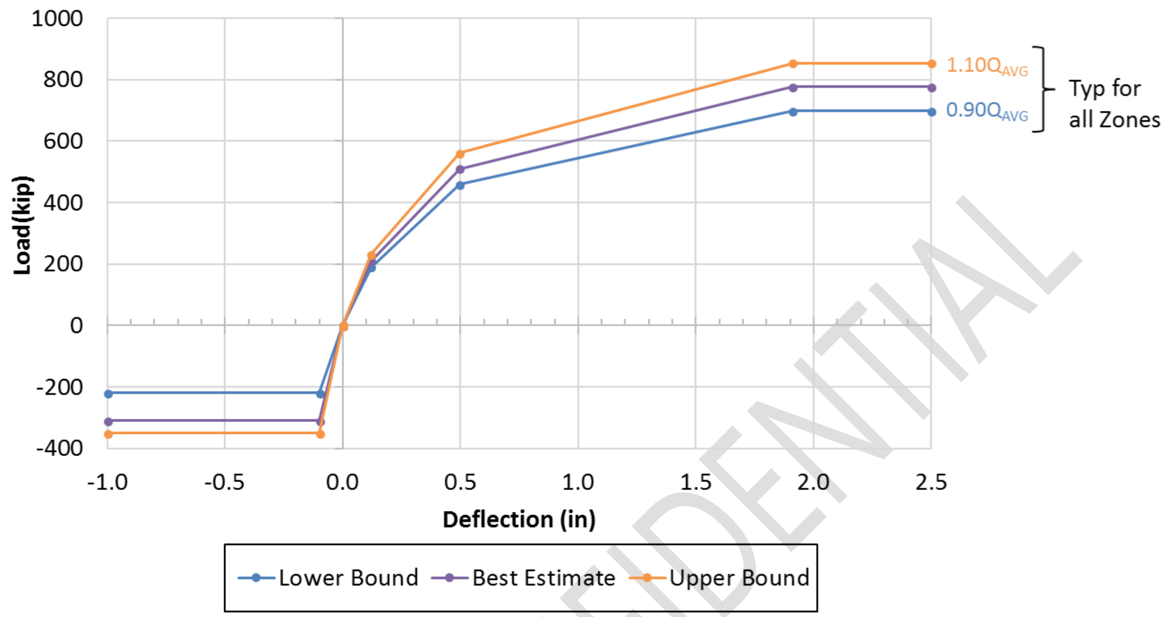


Figure 7-25. Representative Existing Pile Stiffness Bounding (Zone 1)

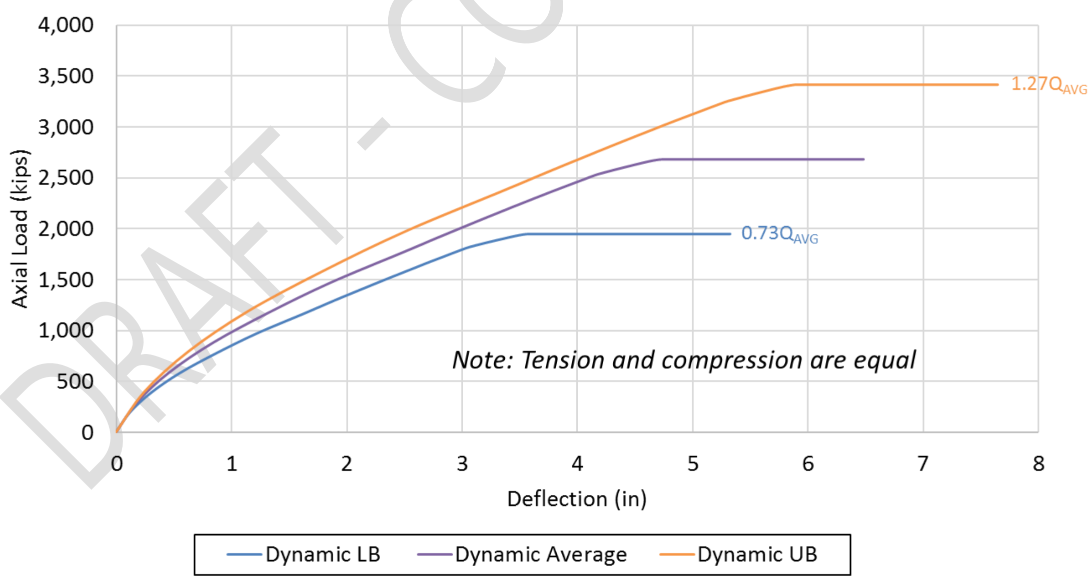


Figure 7-26. Rock Pile Dynamic Stiffness

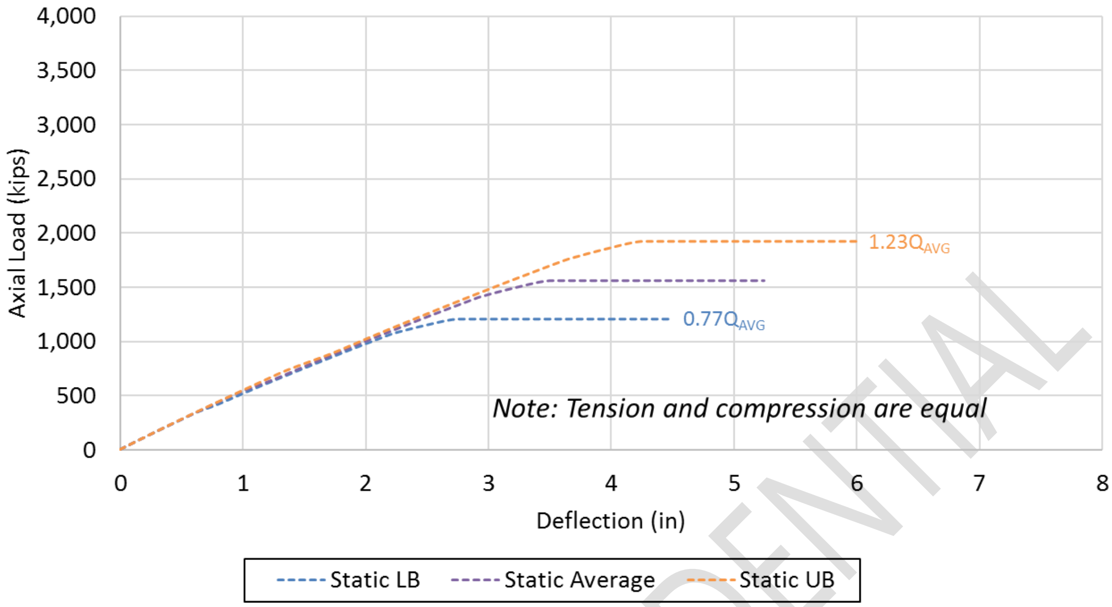


Figure 7-27. Rock Pile Static Stiffness

7.6.2.2. Lateral Pile Stiffness

Lateral spring stiffnesses have been developed using the following procedure:

1) An XTRACT sectional analysis was performed for each pile type to determine moment-curvature (M-Φ) relationships for the expected range of axial loads.
2) The site geotechnical characteristics were analyzed (by ENGEO) to determine upper and lower bound p-y springs for each soil strata within the depth of the pile.
3) LPILE analyses were conducted to using the M-Φ data and p-y springs to determine a lateral load vs. displacement relationship for the same range of imposed axial loads and initial pile head rotations.

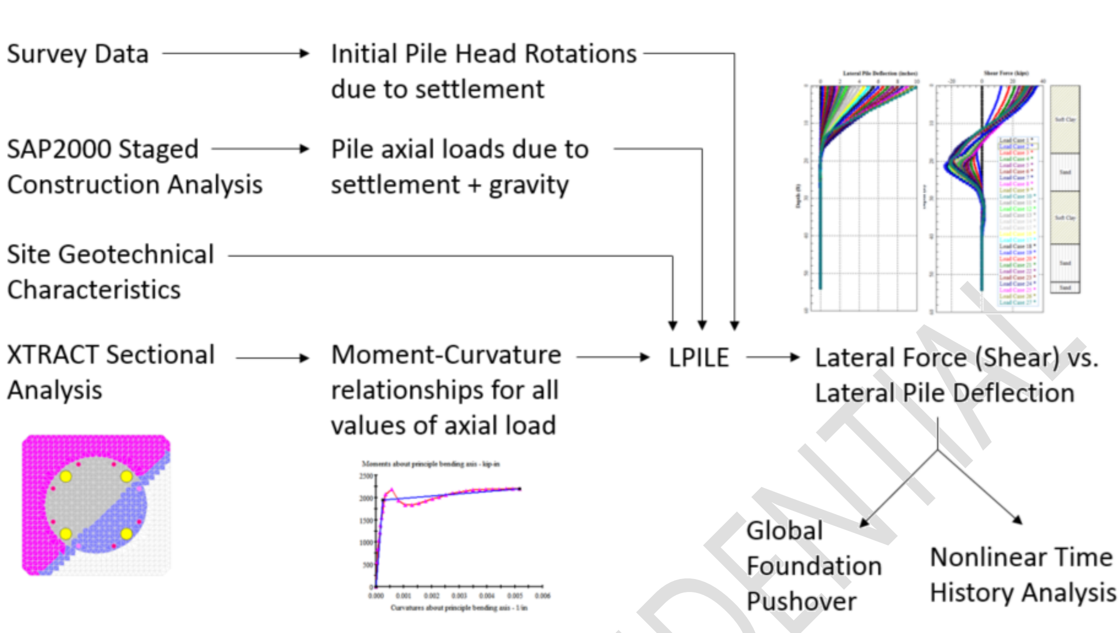


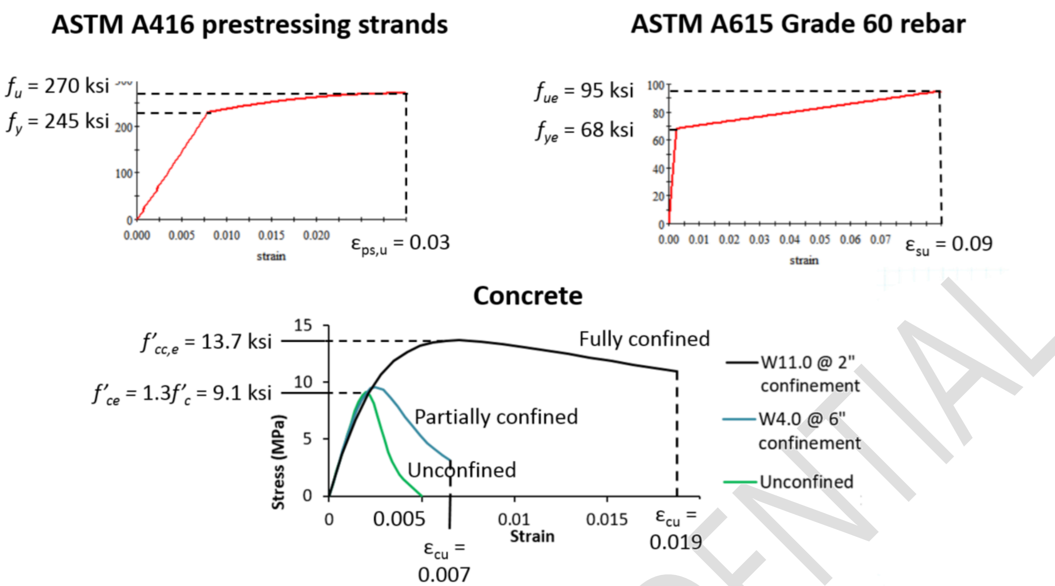
Figure 7-28. Development of Lateral Pile Stiffnesses

Lateral stiffness and displacement limits are functions of the axial load in the element. Since it is not feasible to vary the lateral stiffness as a function of the axial load at each time step in the response history analyses, backbone curves have been assigned for each individual element based on their recorded gravity load at the construction stage being analyzed.

Details of the lateral stiffness parameters for each pile type follow.

<u>Existing Piles</u>

XTRACT models were developed using the material models shown in Figure 7-29. Since the longitudinal rebar and confinement tie spacing vary along the height of the piles (see Figure 7-30), separate M-Φ relationships were determined for the top, middle, and bottom of the piles (Figure 7-31).



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 7-29. Existing Pile Material Models for Sectional Analysis

A representative example of the LPILE analysis procedure and results is shown in Figure 7-32. In all, a suite of 10,000+ LPILE analyses were run to capture every potential combination of the following variables:

1) Axial Load in the pile (250k Tension to 800k Compression)
2) Soil Properties (Lower Bound, Average, or Upper Bound)
3) Initial Pile Head Rotation (-0.50°, -0.25°, 0°, +0.25°, +0.50°)

A few representative examples of the impact of these variables is shown in the following figures. The LPILE results at each axial load increment are provided in Figure 7-33 (for average soil properties with 0° initial pile head rotation). An example of the LPILE results using upper and lower bound soil properties is provided in Figure 7-34 (for axial load, P = 600 kips with 0° initial pile head rotation). An example of the pile lateral capacity degradation by initial rotation is provided in Figure 7-35.

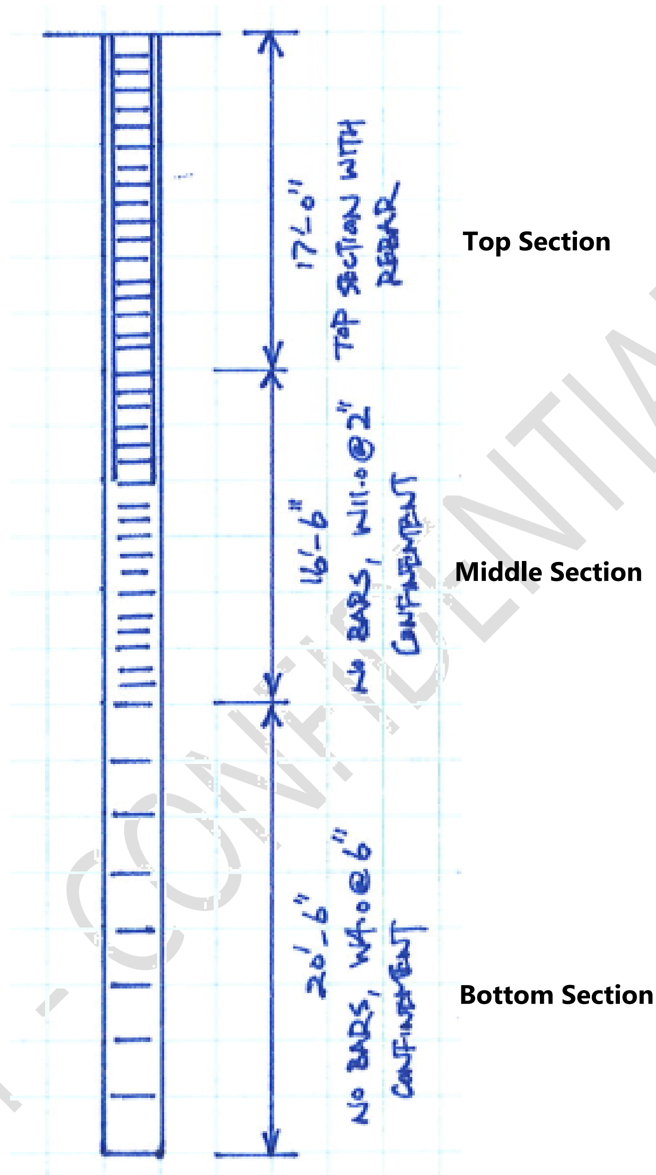


Figure 7-30. Theoretical Existing Pile Reinforcement Zones

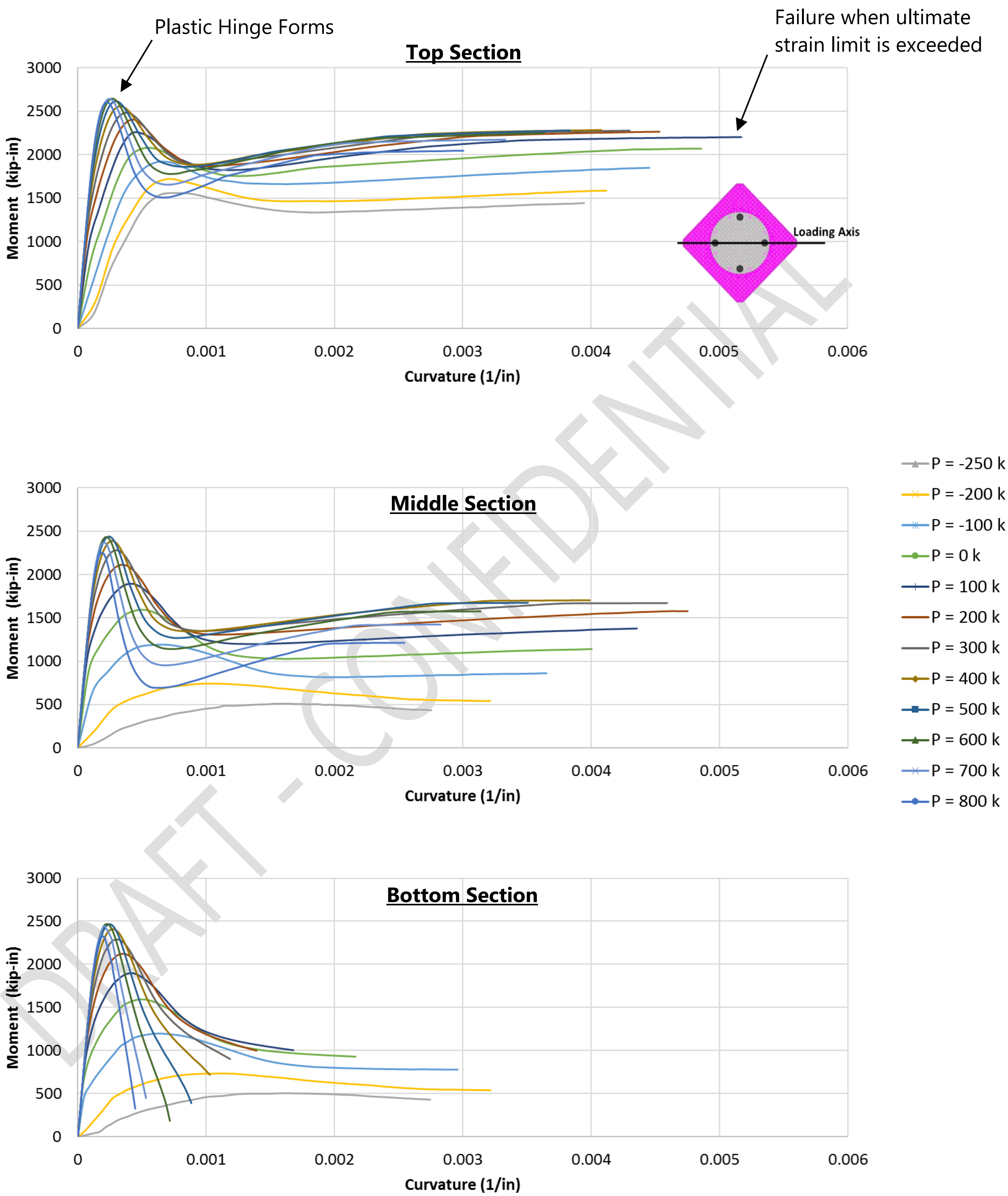
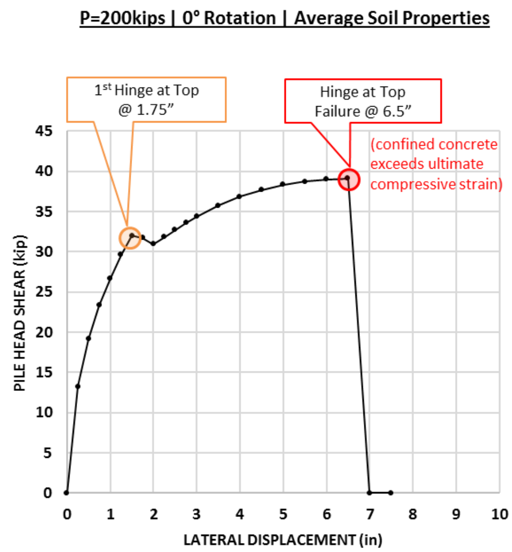
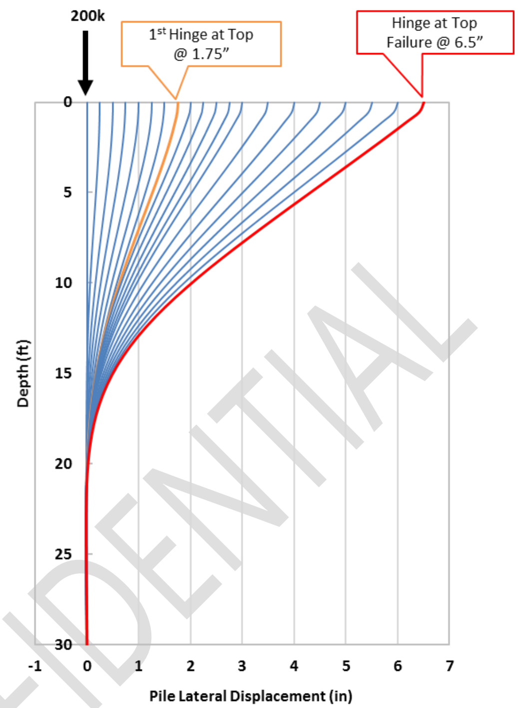


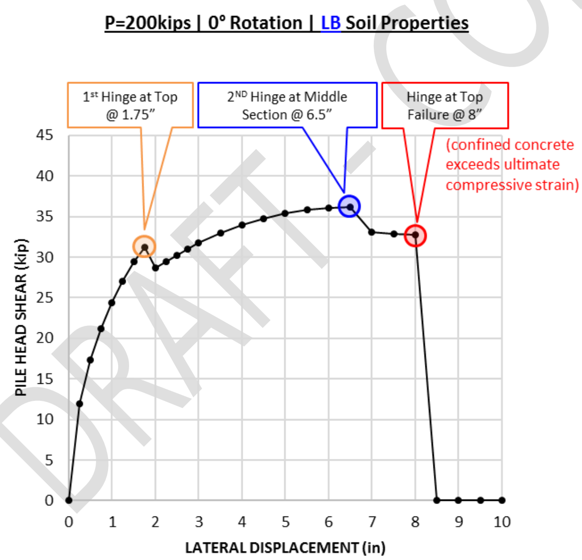
Figure 7-31. Existing Pile Moment-Curvature Relationships

CASE 1





CASE 2



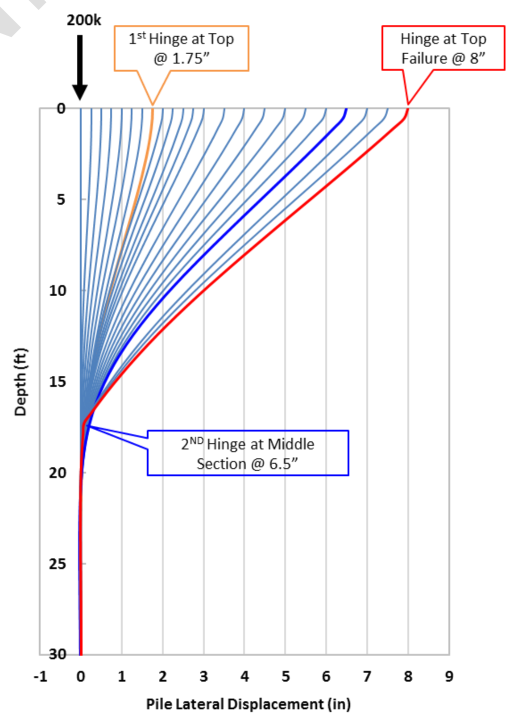


Figure 7-32. Representative Example of LPILE Analysis (Axial Load, P = 200 kips)

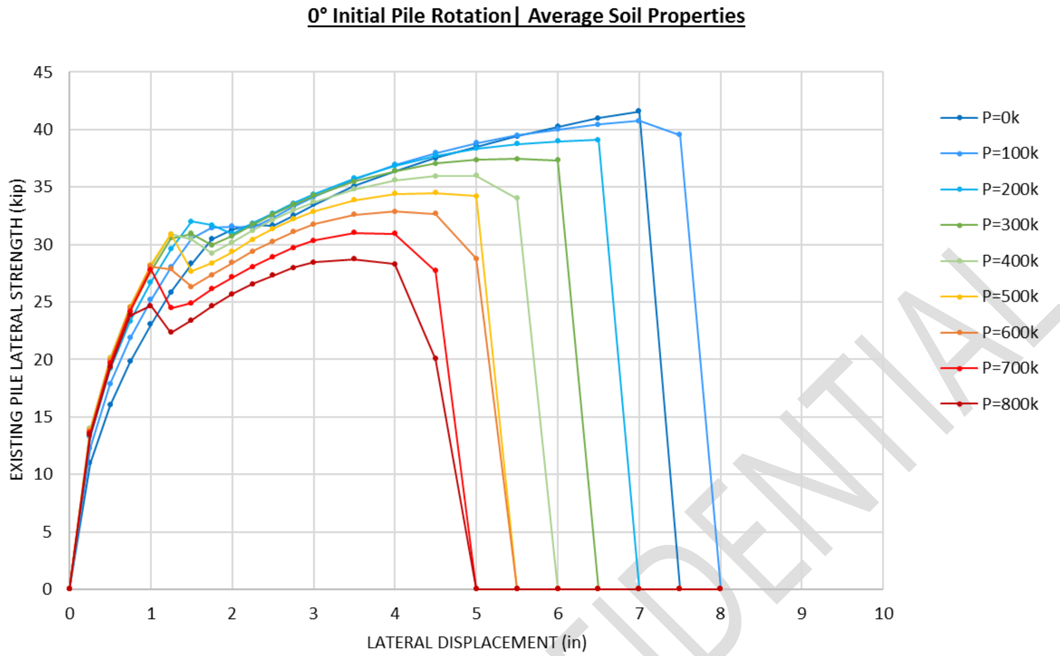


Figure 7-33. Existing Pile Lateral Stiffness Variation by Axial Load

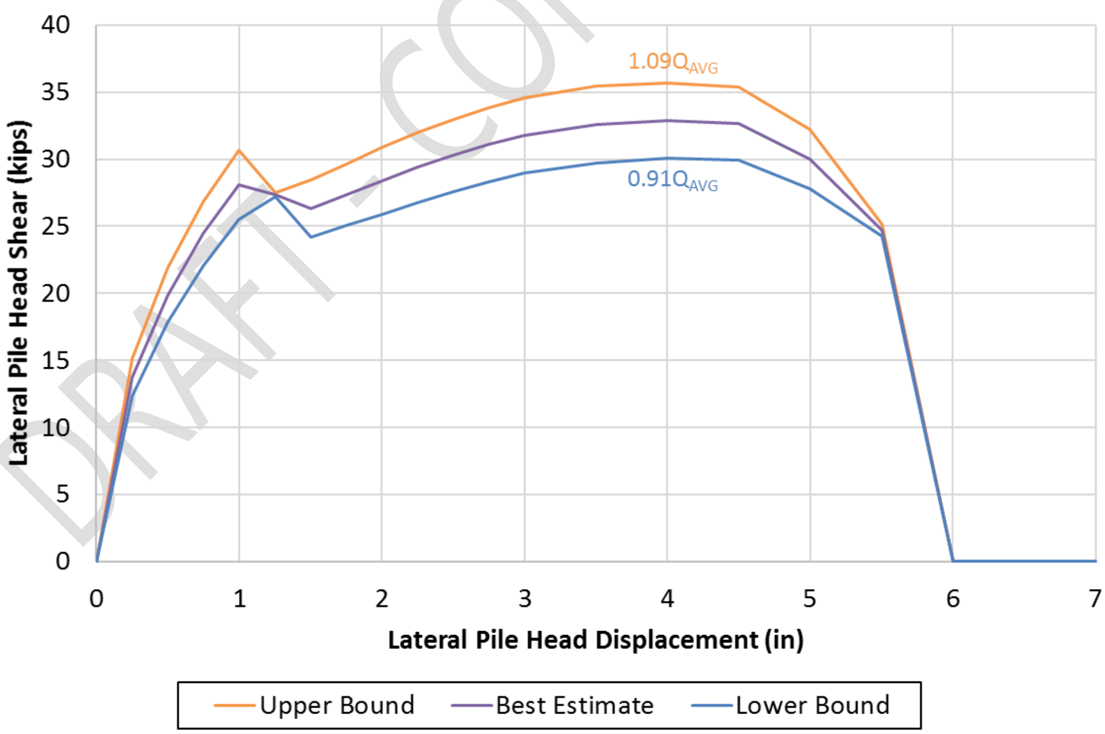


Figure 7-34. Existing Pile Lateral Stiffness Variation by Soil Stiffness Bounding

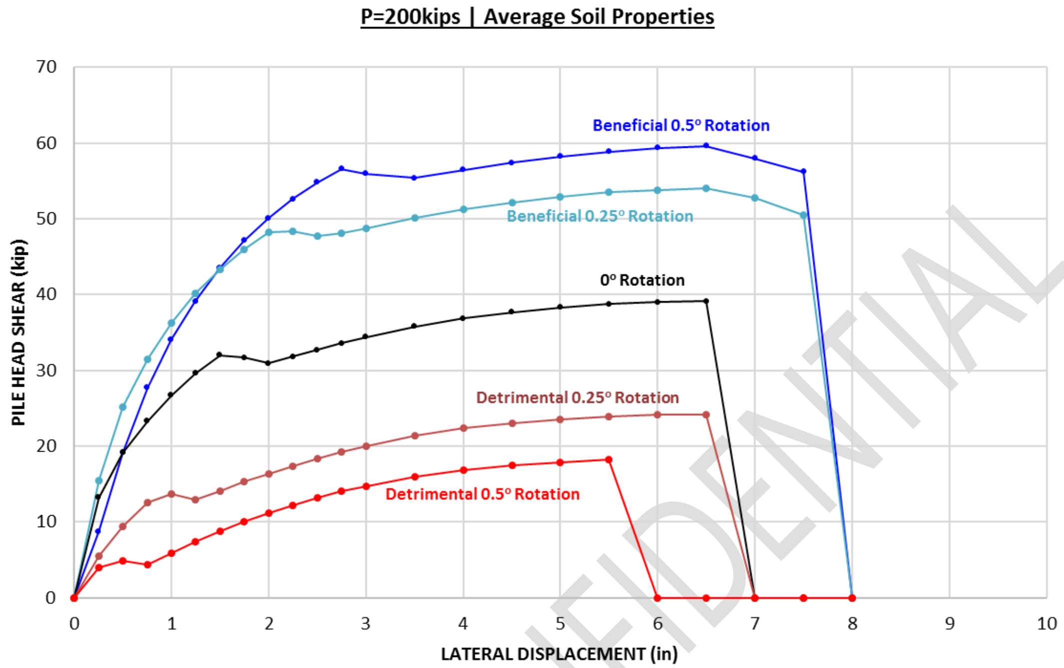


Figure 7-35. Existing Pile Lateral Stiffness Variation by Initial Pile Head Rotation

New Rock Piles

The same procedure described above was repeated for the new rock piles. XTRACT models were developed using the material models shown in Figure 7-36. Moment-curvature relationships were developed at the upper and lower sections of the pile (Figure 7-37).

Since the new piles will not be subjected to initial pile head rotations, the suite of LPILE analyses was reduced. For the new piles, LPILE analyses were run to capture every potential combination of the following variables:

1) Axial Load in the pile (1500k Tension to 2500k Compression)
2) Soil Properties (Lower Bound, Average, or Upper Bound)

A few representative examples of the impact of these variables is shown in the following figures. The LPILE results at each axial load increment are provided in Figure 7-38 (for average soil properties). An example of the LPILE results using upper and lower bound soil properties is provided in Figure 7-39 (for axial load, P = 1000 kips).

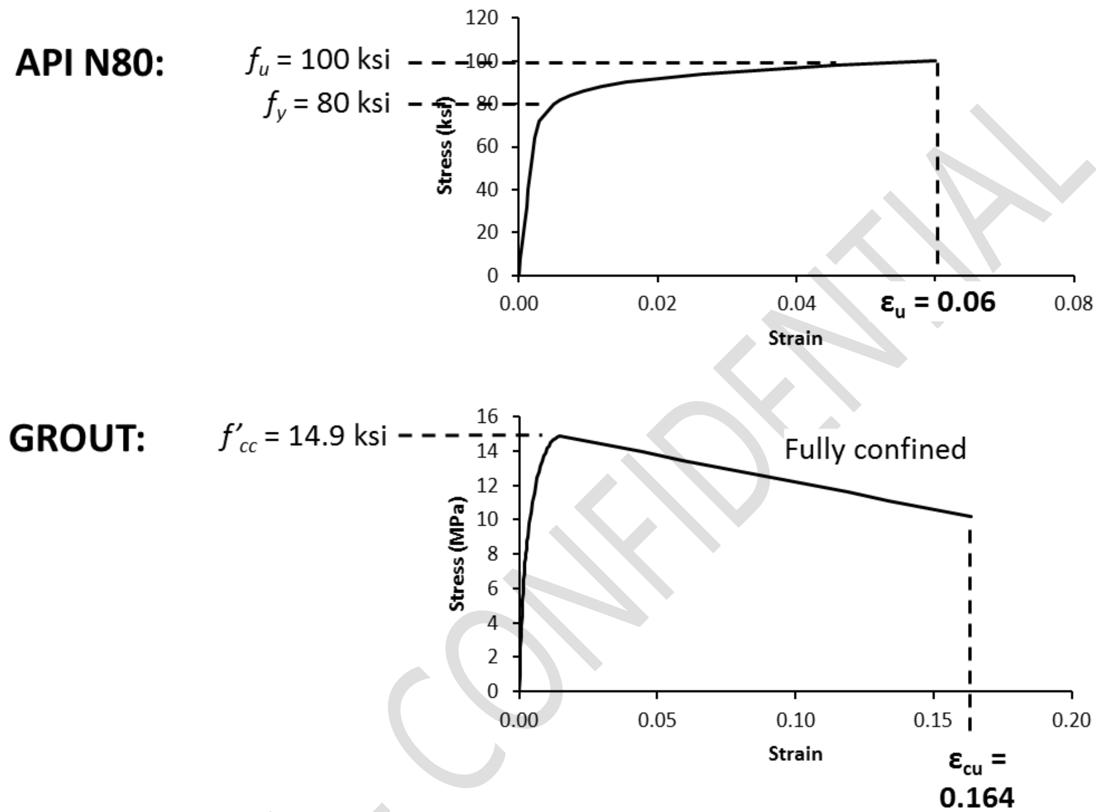
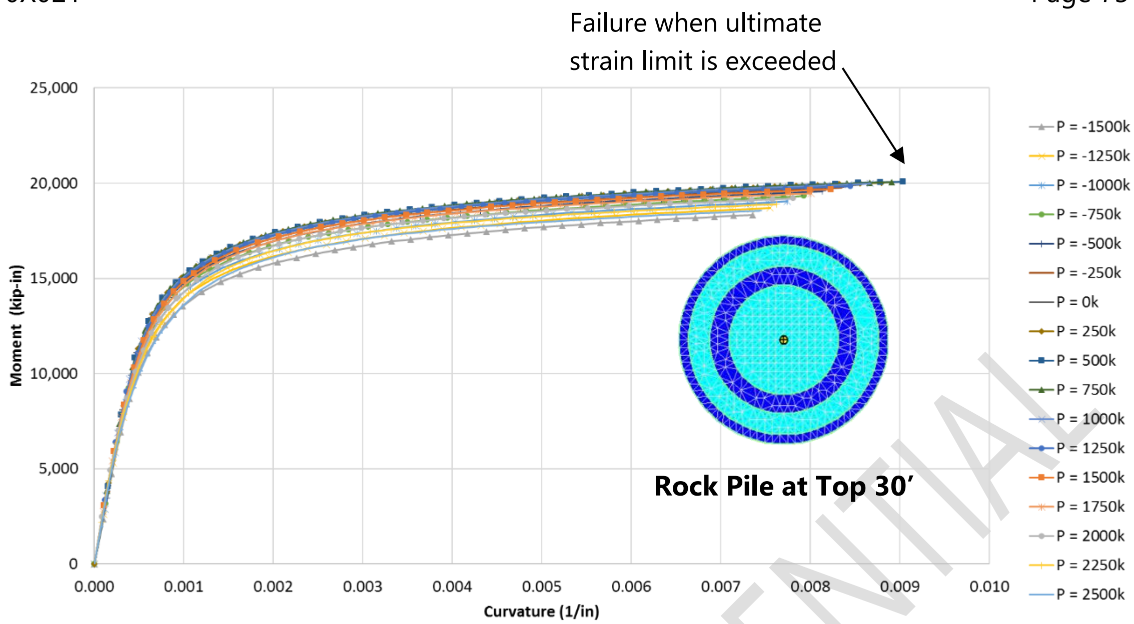


Figure 7-36. New Pile Material Models for Sectional Analysis



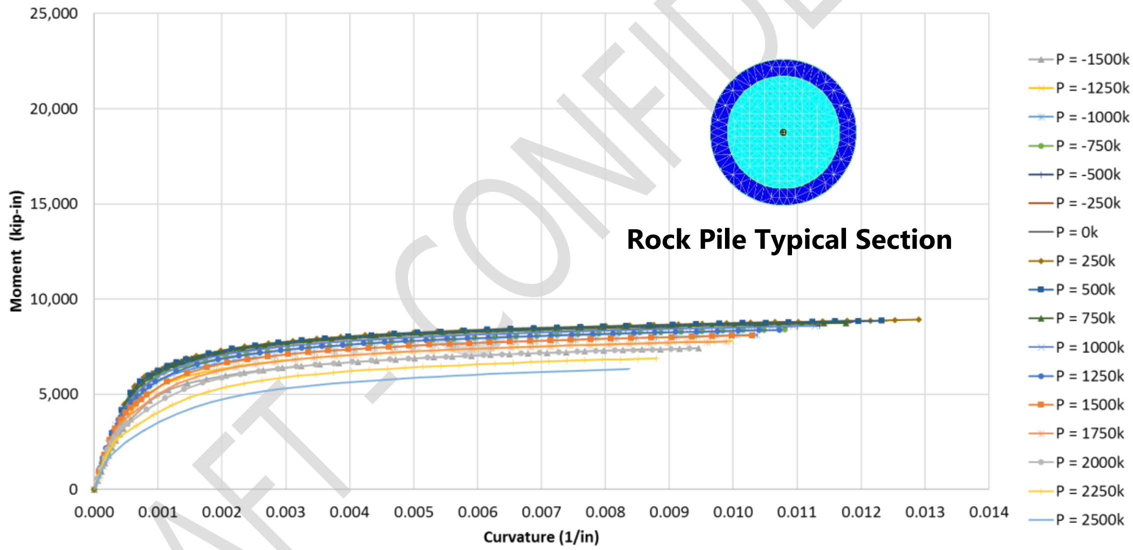


Figure 7-37. New Pile Moment-Curvature Relationships

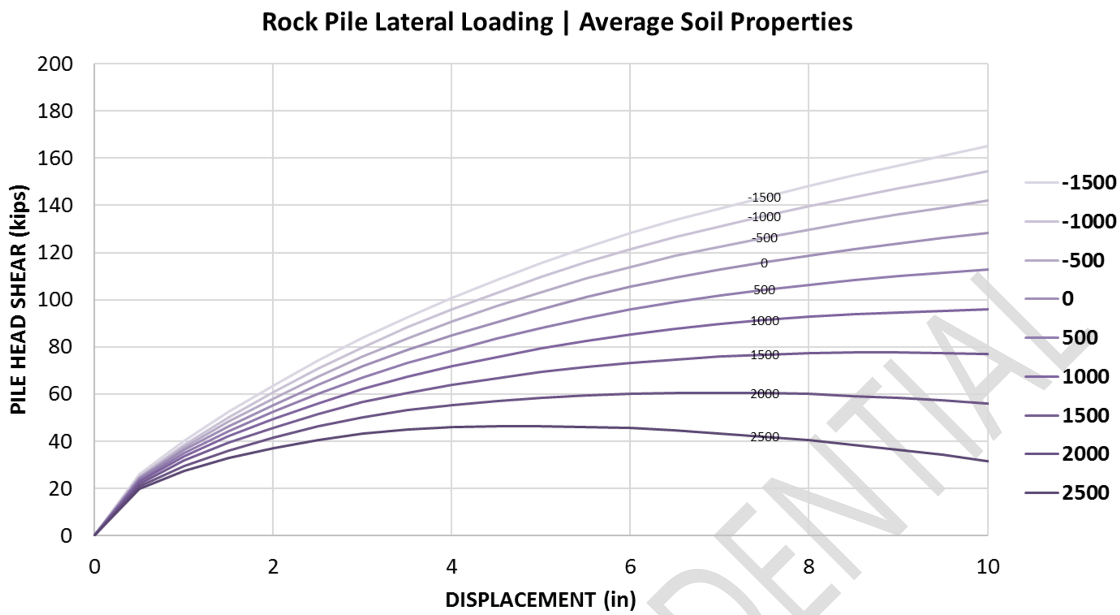


Figure 7-38. New Pile Lateral Stiffness Variation by Axial Load

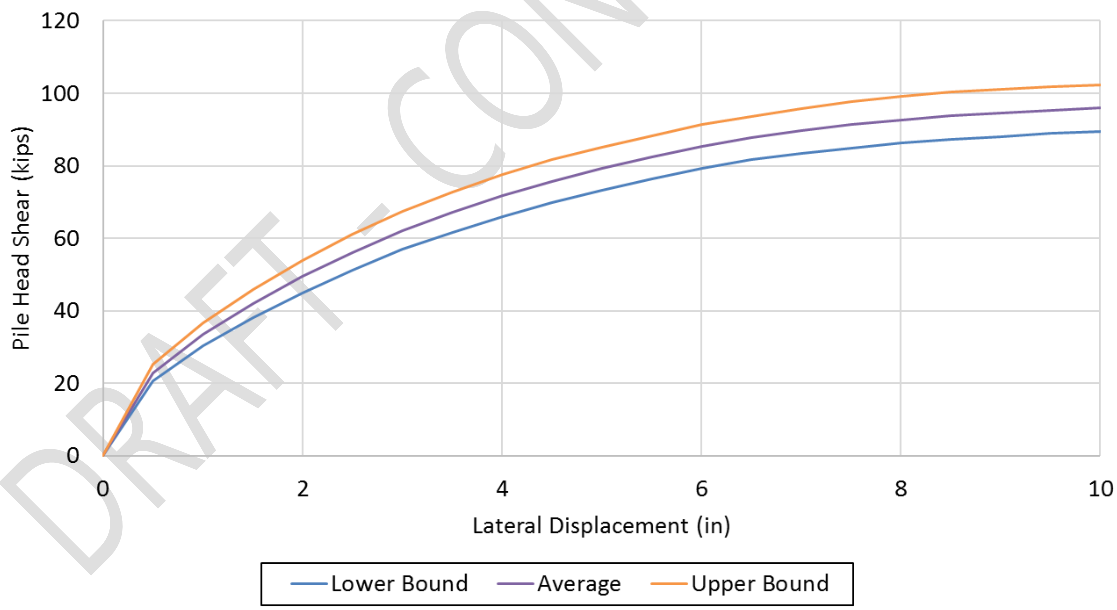


Figure 7-39. New Pile Lateral Stiffness Variation by Soil Stiffness Bounding

Note that the pile lateral bending behavior is affected by the location and design of the casing threaded splices. At these splice locations the stiffness and ductility is reduced.

As there isn't testing data publicly available that can be used to fairly estimate the amount of reduced stiffness and ductility we can expect for our piles, project specific bending testing is required. These bending tests will be accomplished for sample Rock piles with varying splice thread lengths to establish expected performance for different designs. The final splice design and allowable splice locations will be informed by this testing. Once a final splice design is decided, the analysis will be updated to account for the expected behavior. To estimate the final behavior, a modest amount of reduced tensile strength and stiffness was assumed in arriving at the load-displacement curves in Figure 7-38 and Figure 7-39. This work is ongoing.

8 GRAVITY LOADING CRITERIA

The gravity loading criteria are listed in Table 8-1. The loads are in addition to the self-weight of the structure. For the determination of the self-weight of the structure, normal weight concrete is taken at a unit weight of 150 pcf, and lightweight concrete is taken at a unit weight of 117 pcf. Live loads, LL, are reducible except where noted NR (not reducible).

Note that the superimposed dead loads, SDL, are taken from the original foundation submittal (p1.7-1 & 2), except they have been modestly increased to account for a more accurate take-off of the interior partitions shown in the Architectural drawings, see section 8.1.

8.1. Partition Load Take-Off

We have calculated partition loads that are approximately twice as heavy as the design allowance per the original foundation calculation submittal (p1.7-1). The design allowance for partitions was 6 psf. Our take-off calculates 10-12 psf. We have used 10 psf in our analyses.

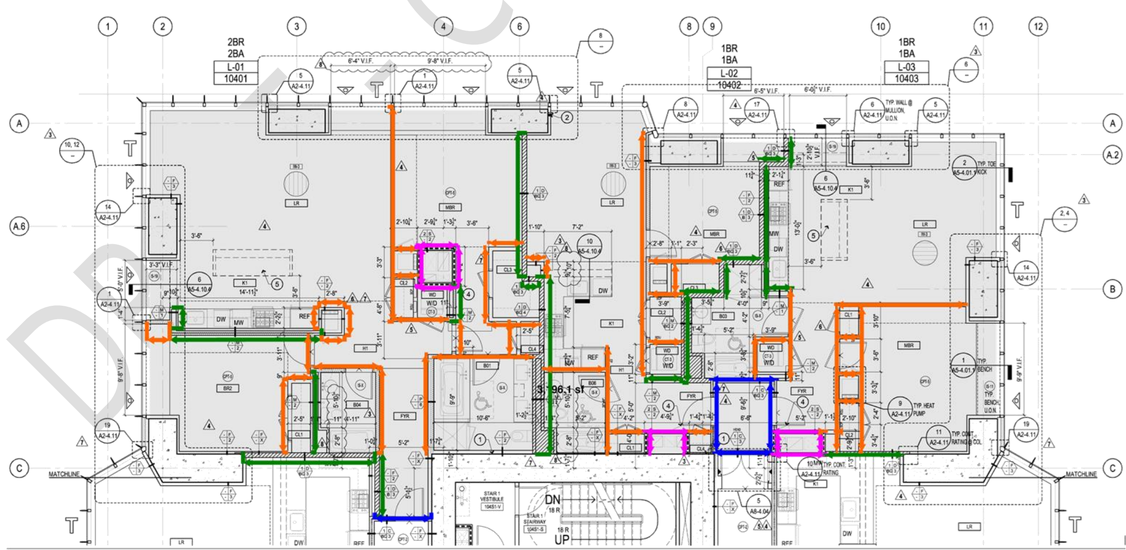


Figure 8-1. Partition Layout for Representative Residence

Table 8-1. Gravity loading

Occupancy	SDL (psf)	LL (psf)
Roof		
Roofing	5	
Pavers	25	
MEP (window washing equip)	15	
Miscellaneous	5	
Exterior Walls	10	
Total	**60**	**20**
Typical Residential Floor		
Flooring	4	
Partition Walls	10[Note 1]	
MEP	3	
Miscellaneous	3	
Exterior Walls	5	
Total	**25**	**40**
Mechanical Floor		
Concrete pads	6	
MEP equipment	20	
Typical MEP	3	
Exterior Walls	5	
Total	**34**	**75**
Ground Floor Level		
Flooring	25	
Partition Walls	25	
MEP	5	
Miscellaneous	5	
Exterior Walls	5	
Total	**75**	**100**

1. Partition loads on the typical residential floors have been increased from 6psf noted in original foundation submittal to 10psf to account for actual partition weight. See Section 8.1.

9 SEISMIC DESIGN CRITERIA

The seismic building parameters are shown in Table 9-1.

Table 9-1. Building parameters

Parameter	**Value**
Risk category	II
Seismic importance factor	I_e = 1.0
Site class	D
Seismic design category	D
Response modification factor[Note1]	R = 7
Deflection amplification factor	C_d = 5.5
Redundancy factor	ρ = 1.0
Fundamental Building Period [Note2]	T = 4.5s

1. R-factor is for a dual-system with special reinforced concrete shear walls and special moment frame capable of resisting at least 25% of prescribed seismic force.
2. 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.

The mapped spectral acceleration parameters for the project site are shown in Table 9-2. These values have been obtained from the USGS US Seismic Design Maps application (https://earthquake.usgs.gov/designmaps/us/application.php) for the project site for 2015 IBC.

Table 9-2. Mapped acceleration parameters

Parameter	**Value**
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

For purposes of ground motion selection and scaling, the period range of interest for the building is established in accordance with the PEER *Guidelines* and ASCE 7-16 Chapter 16. The lower bound of the period range is defined as the period at which the building achieves 90% mass participation which occurs at a period of 0.3s. The upper bound is defined as 2.0T (2.0 x 4.5s = 9.0s). Therefore, the period range of interest is between 0.3s and 9.0s.

9.1. Site-Specific Seismic Hazard Analysis

A site-specific seismic hazard analysis has been performed by ENGEO. The site-specific design basis earthquake (DBE), service-level, and risk-targeted maximum considered earthquake (MCE_R) spectra recommended by ENGEO are shown in Table 9-3 and Figure 9-1.

Table 9-3. Site-specific spectral accelerations recommended by ENGEO

Period (s)	DBE S_a (g)	SLE S_a (g)	MCE_R S_a (g)
0.01	0.363	0.174	0.545
0.02	0.400	0.760	0.600
0.03	0.440	0.188	0.660
0.05	0.541	0.236	0.811
0.075	0.624	0.311	0.936
0.1	0.720	0.418	1.080
0.15	0.800	0.554	1.200
0.2	0.953	0.625	1.429
0.25	0.994	0.644	1.491
0.3	1.007	0.632	1.511
0.4	0.931	0.561	1.396
0.5	0.882	0.591	1.323
0.6	0.800	0.429	1.200
0.75	0.640	0.336	0.960
1	0.511	0.235	0.767
1.5	0.497	0.136	0.745
2	0.311	0.088	0.467
3	0.171	0.046	0.256
4	0.121	0.029	0.181
5	0.102	0.019	0.153
6	0.082	0.015	0.123
7	0.070	0.012	0.105
8	0.060	0.009	0.090
9	0.053	0.007	0.080
10	0.048	0.005	0.072

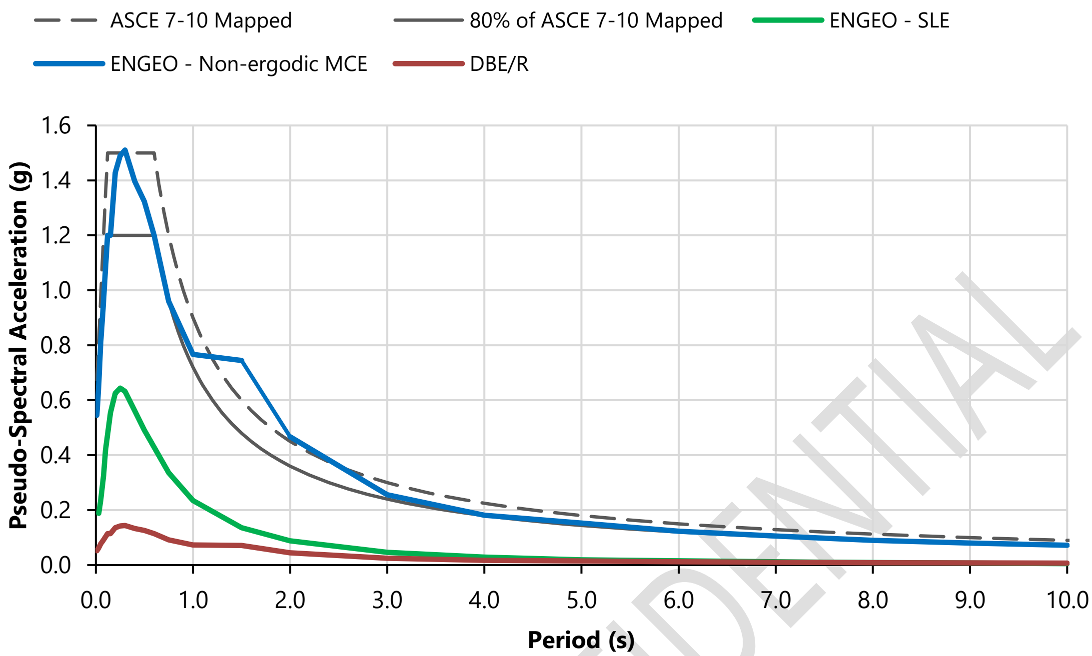


Figure 9-1. Site-specific spectral acceleration spectra recommended by ENGEO

The site-specific design acceleration parameters are determined in accordance with ASCE 7-10 §21.4 and are listed in Table 9-4.

Table 9-4. Site-specific design acceleration parameters

Parameter	Value
Site-specific MCE_R spectral acceleration at short periods	S_{MS} = 1.429 g
Site-specific MCE_R spectral acceleration at a period of 1 s	S_{M1} = 0.934 g
Site-specific DBE spectral acceleration at short periods	S_{DS} = 0.953 g
Site-specific DBE spectral acceleration at a period of 1 s	S_{D1} = 0.511 g

9.2. Service Level Ground Motion Records

The service-level ground motion records recommended by ENGEO for use in the response history analyses are listed in Table 9-5, and the pseudo-acceleration spectra for the ground motion records are shown in Figure 9-2 and Figure 9-3. ENGEO provided nine ground motions instead of the minimum seven to include sufficient long duration motions and meet the other required code criteria.

Noted angles of application are for the two horizontal components of the ground motions and have been applied to the building measured counterclockwise with respect to the project East direction.

Table 9-5. Service-level ground motion records recommended by ENGEO

No.	Earthquake	NGA #	Pulse Period (s)	Magnitude (M_W)	SF	Angle of Application (°)
1	Coalinga	357	-	6.36	1.67	99/189
2	Northridge	1035	-	6.69	1.11	0/90
3	Hector Mine	1794	-	7.13	0.73	173/263
4	Chi-Chi	3268	-	6.30	0.83	174/264
5	Cape Mendocino	3751	-	7.01	0.70	29/119
6	Parkfield	4078	-	6.0	1.94	90/180
7	Chuetsu-oki	4858	-	6.8	0.80	175/265
8	Darfield	6891	-	7.0	1.02	173/263
9	Christchurch	8069	-	6.2	1.91	88/178

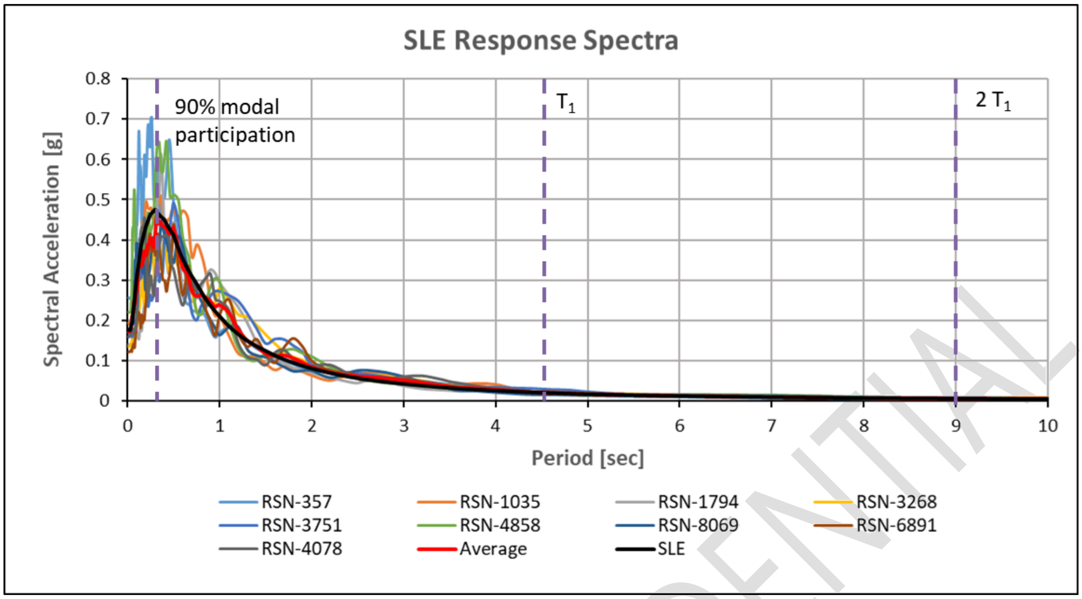


Figure 9-2. Service-level ground motions – individual scaled RotD100 acceleration spectra

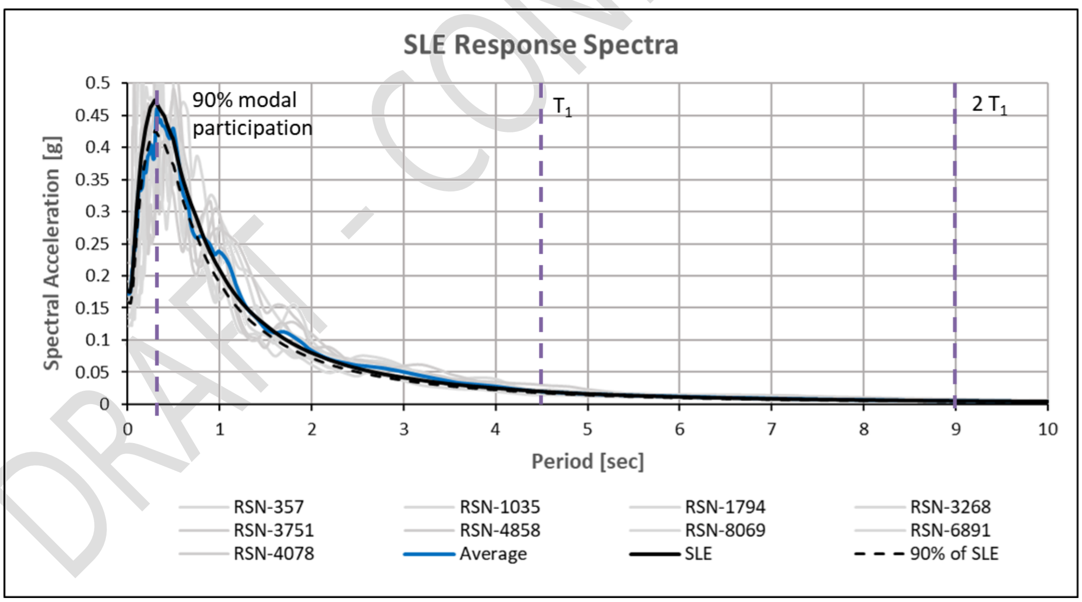


Figure 9-3. Service-level ground motions – average scaled RotD100 acceleration spectrum

9.3. MCE_R Ground Motion Records

The MCE ground motion records recommended by ENGEO for use in the response history analyses are listed in Table 9-6, and the pseudo-acceleration spectra for the ground motion records are shown in Figure 9-4 and Figure 9-5.

Noted angles of application are for the two horizontal components of the ground motions and have been applied to the building measured counterclockwise with respect to the project East direction. Rotation of the ground motions is not required because our site does not meet the near-fault criteria.

Table 9-6. MCE ground motion records recommended by ENGEO

No.	Earthquake	NGA #	Pulse Period (s)	Magnitude (M_W)	SF	Angle of Application (°)
1	Imperial Valley	178	4.5	6.53	1.88	22/112
2	Imperial Valley	184	6.2	6.53	2.20	90/180
3	Westmorland	316	4.3	5.9	2.04	173/263
4	Loma Prieta	802	4.5	6.93	1.88	0/90
5	Landers	832	-	7.28	3.66	62/152
6	Kocaeli	1163	-	7.51	3.83	106/196
7	Chi-Chi	1261	-	7.62	4.00	41/131
8	Chi-Chi	1511	4.7	7.62	1.50	136/226
9	El Mayor-Cucapah	5827	-	7.2	1.50	46/136
10	Darfield	6890	-	7.0	2.50	92/182
11	Darfield	6959	12.0	7.0	1.21	90/180

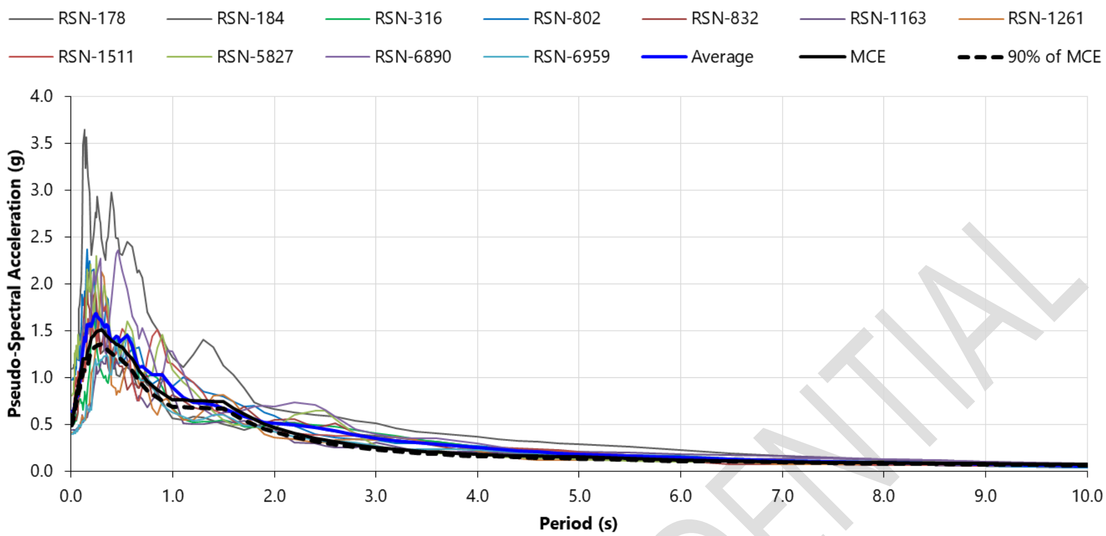


Figure 9-4. MCE ground motions – individual scaled RotD100 spectra

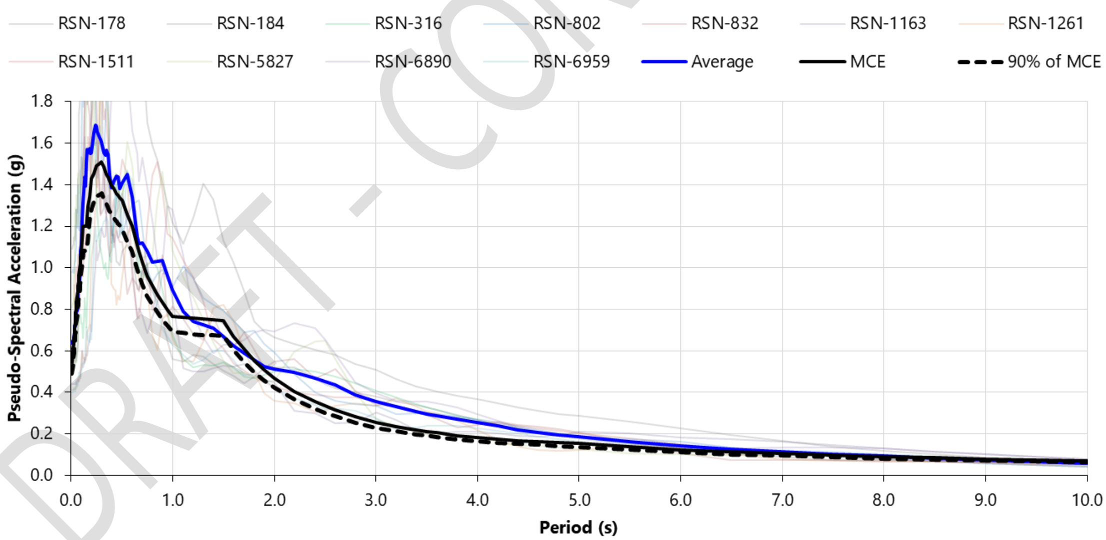


Figure 9-5. MCE ground motions – average scaled RotD100 spectrum

10 WIND LOADING CRITERIA

The wind load parameters are shown in Table 10-1. Calculated wind loads per the Directional Procedure of ASCE 7-10 are shown in Figure 10-1. The wind loads in the East-West and North-South directions are applied as described in Figure 10-2, per ASCE 7-10 §27.4.6.

Table 10-1. Wind parameters

Parameter	Value
Risk category	II
Mapped ultimate (700-year) design wind speed	V_{ult} = 110 mph
Exposure	D
Topographic factor	K_{zt} = 1.0
Directionality factor	K_d = 0.85
Gust effect factor[Note 1]	G = 0.97 (E-W) G = 0.99 (N-S)
Enclosure	Enclosed
Internal pressure coefficient	GC_{pi} = ±0.18

1. The gust effect factors are calculated per the requirements of ASCE 7-10 §26.9.5 for flexible buildings using a fundamental natural frequency = 1/4.2 hz = 0.238 Hz in the East-West direction and 1/4.4 Hz = 0.227 hz in the North-South direction.

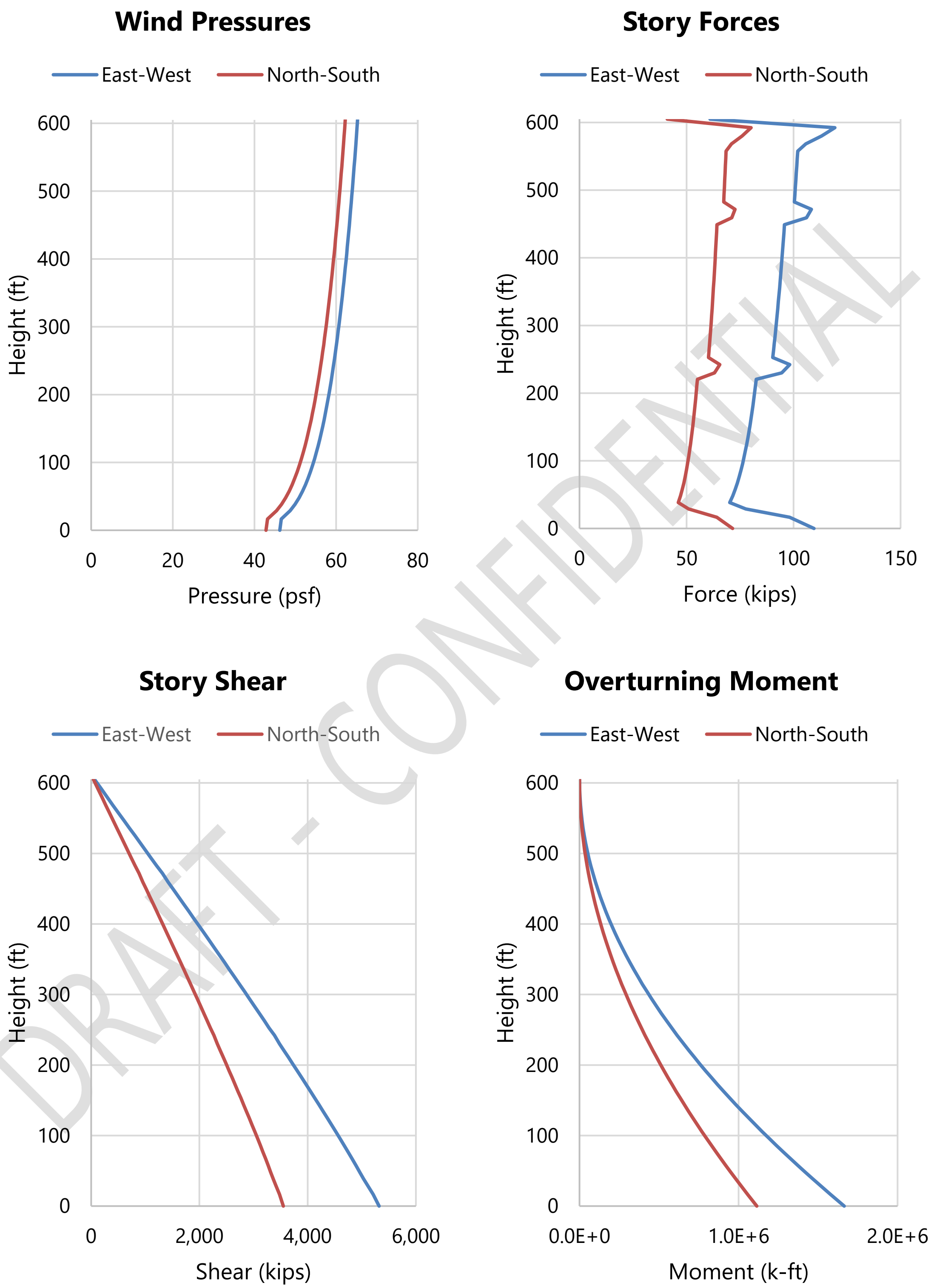
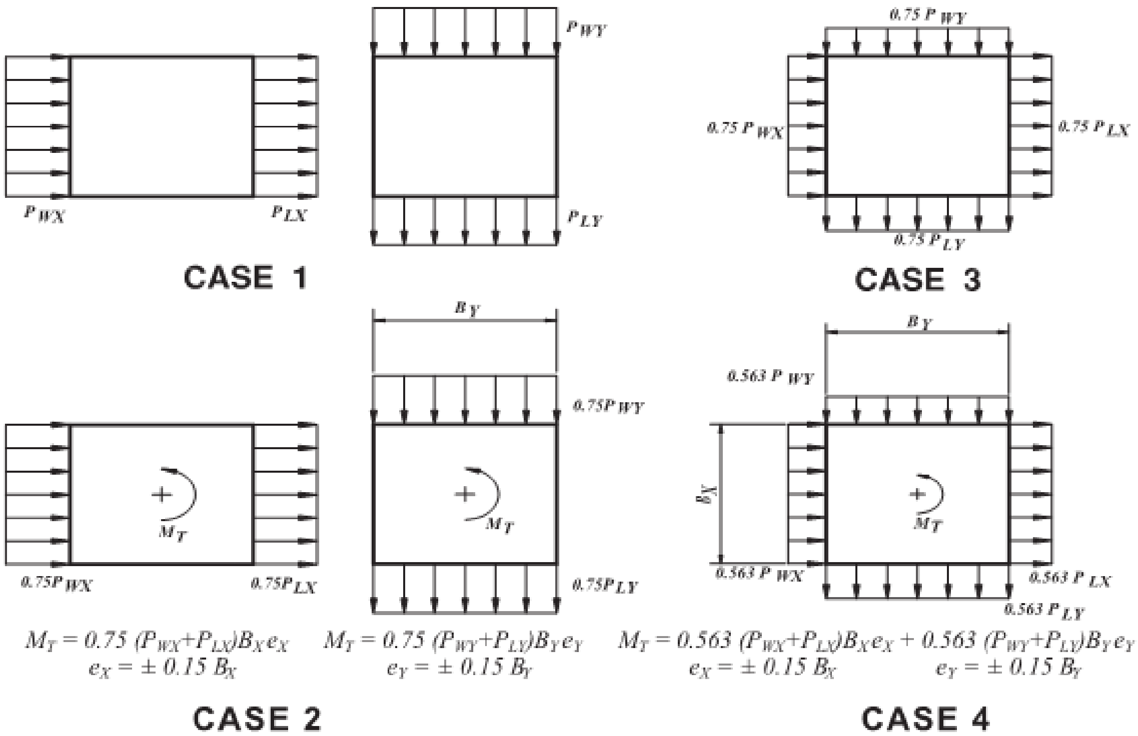


Figure 10-1. Wind Loads Calculated per the ASCE 7-10 Directional Procedure



Notation:

P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

Figure 10-2. Design Wind Load Cases per ASCE 7-10

11 LATERAL EARTH PRESSURES

Due to the Low-Rise basement to the east and the Transbay to the south, unbalanced soil pressures are exerted on the north and west basement walls (Figure 11-1). These unbalanced lateral earth pressures add to the base shears and displacements experienced by the foundation.

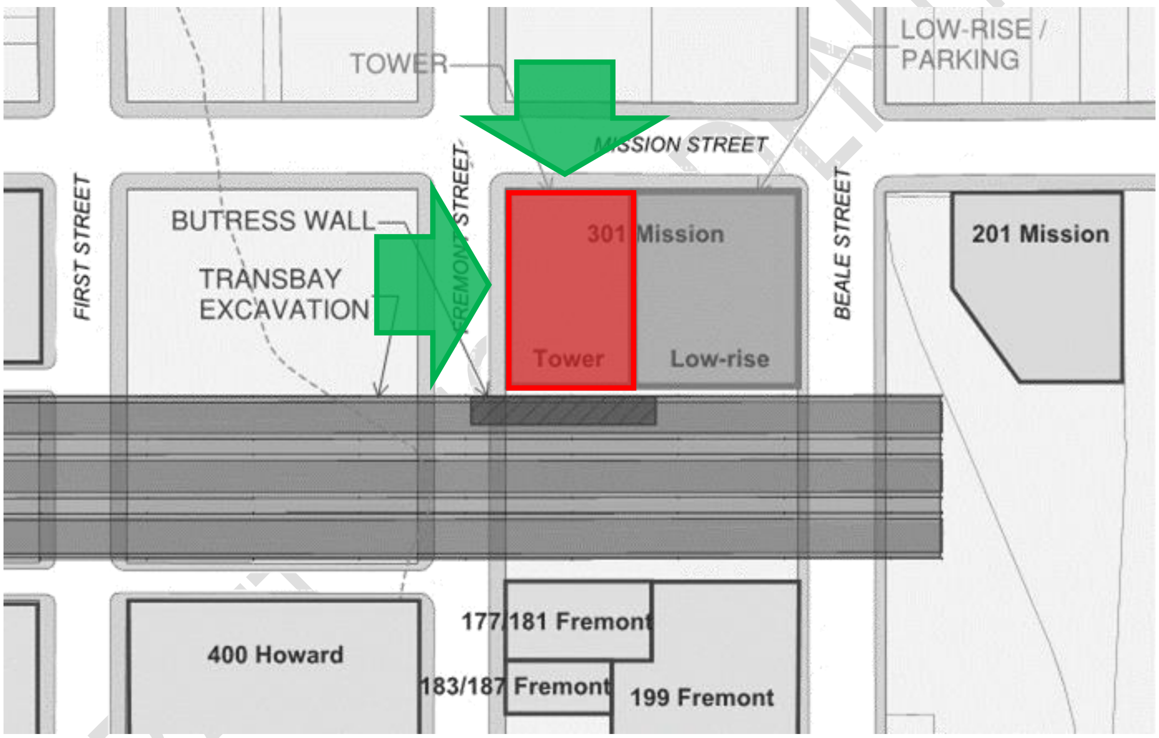


Figure 11-1. Lateral Earth Pressures on Basement walls

11.1. At-Rest Earth Pressures

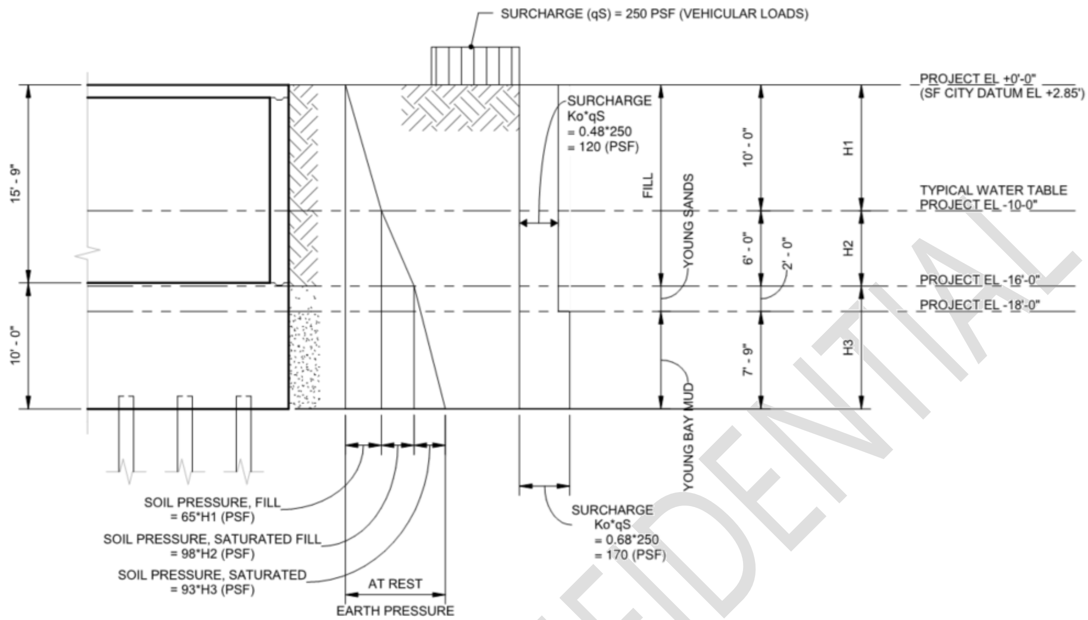


Figure 11-2. Static Lateral Earth Pressures on Basement walls

11.2. Active / Seismic Earth Pressures

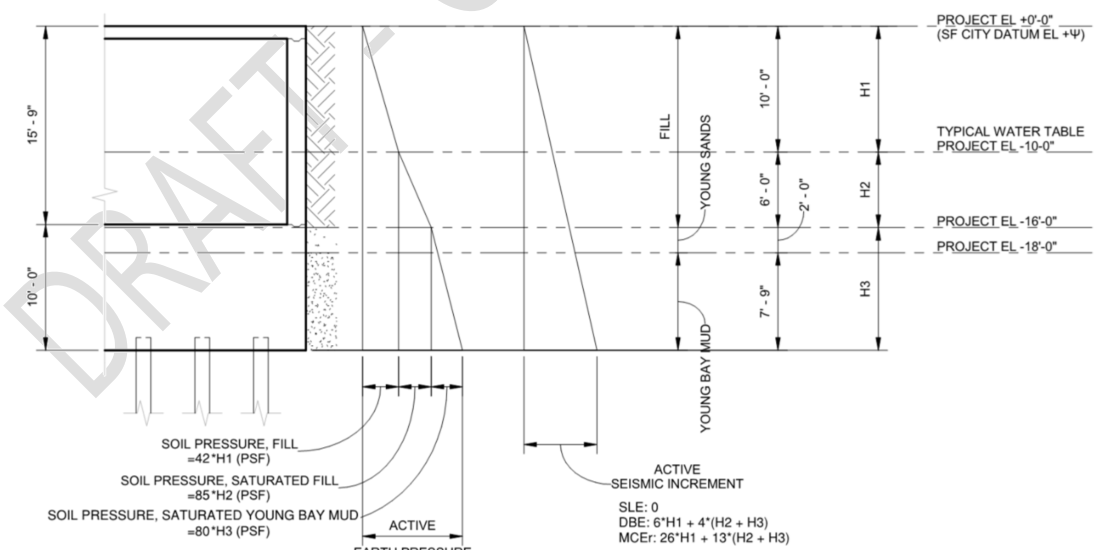


Figure 11-3. Active Lateral Earth Pressures on Basement walls

11.3. Passive Soil Resistance to Lateral Loads

Passive soil resistance to lateral loads is accounted for on basement walls on the north and west sides, and on all sides of the 21′ deep thickened mat section around the elevator pit (See Figure 1-6 and Figure 1-7 for more information on the elevator pits). Recommendations for passive pressure at the transition in mat depth from 10′ to 3′ on the south side are still being developed by ENGEO and have been ignored in the current analysis. Figure 11-4 summarizes the total passive soil resistance in each orthogonal direction.

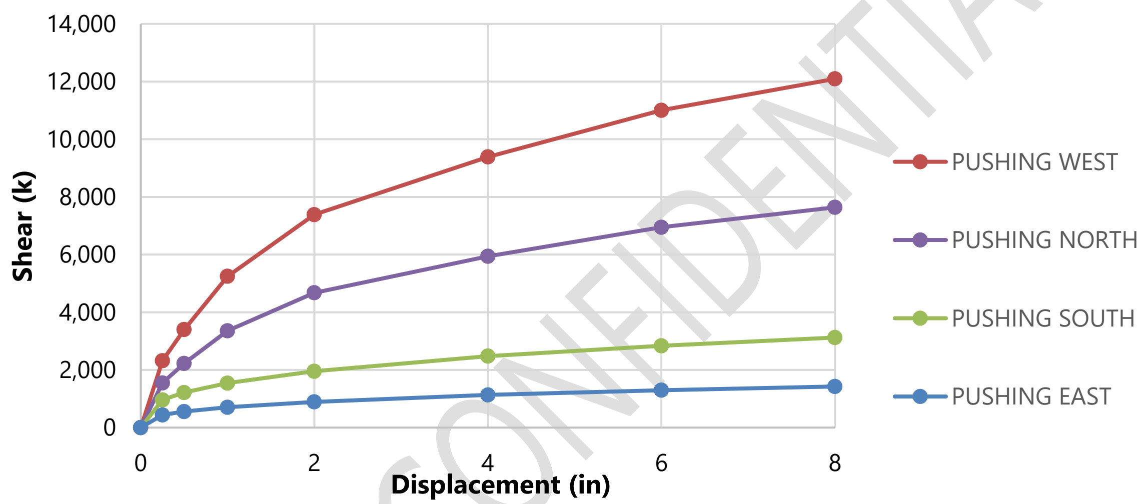


Figure 11-4. Passive Pressure on Basement Walls and Elevator Pit

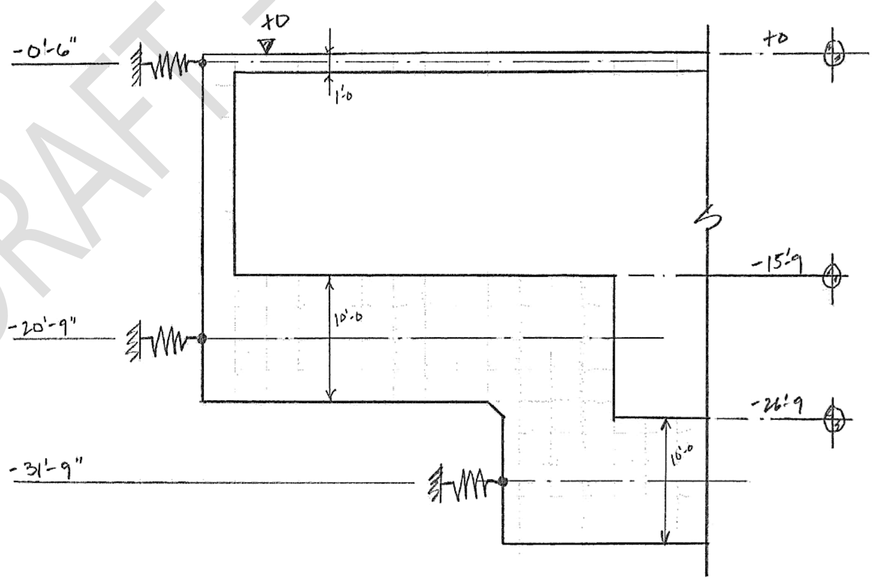


Figure 11-5. Passive Pressure Springs in Analysis Model

12 LOAD COMBINATIONS

12.1. Gravity Loads

Allowable Stress Design (ASD):

The following load combination is used for the Allowable Stress Design of foundation elements per California Building Code §1605.3:

$$D + L$$

Load and Resistance Factor Design (LRFD):

The following load combinations are used for the Ultimate Strength Design of foundation elements per California Building Code §1605.2:

$$1.4D$$

$$1.2D + 1.6L + 0.5(Lr \text{ or } S \text{ or } R)$$

D = dead load, including construction dead load and superimposed dead load

L = reduced live loads, LL, from Table 8-1

L_r = reduced roof live load

S = snow load

R = rain load

12.2. Design Basis Earthquake (DBE)

Allowable Stress Design (ASD):

The following load combinations are used for DBE Allowable Stress Design, per ASCE 7-10 §12.4.2.3 (with coefficients for H coming from Building Code Section 1605.3.1):

$$(1.0 + 0.14S_{DS})D + 0.7Q_E + \gamma_H H$$

$$(1.0 + 0.10S_{DS})D + 0.75L + 0.75S + 0.525Q_E + \gamma_H H$$

$$(0.6 - 0.14S_{DS})D + 0.7Q_E + \gamma_H H$$

<u>Load and Resistance Factor Design (LRFD):</u>

The following load combinations are used for DBE Strength Design, per ASCE 7-10 §12.4.2.3 (as referenced by California Building Code §1613.1):

$$(1.2 + 0.2S_{DS})D + f_1L + 0.2S + Q_E + \gamma_H H$$

$$(0.9 - 0.2S_{DS})D + Q_E + \gamma_H H$$

S_{DS} = design spectral acceleration at short periods
D = dead load, including construction dead load and superimposed dead load
f_1 = companion load factor on live load, equal to:
= 1.0 for places of public assembly, live loads in excess of 100 psf and parking garages, or
= 0.5 for other live loads
L = reduced live loads, LL, from Table 8-1
S = snow load
Q_E = earthquake effects from response spectrum analysis
γ_H = load factor on earth pressures, equal to:
= 1.0 for ASD, 1.6 for LRFD if the effect of H adds to the seismic load effect,
= 0.6 for ASD, 0.9 for LRFD if the effect of H resists the seismic load effect and H is permanent, or
= 0 if the effect of H resists the seismic load effect and H is not permanent
H = earth pressure

12.3. Service Level Earthquake (SLE)

The following load combination is used for nonlinear response history analysis with each pair of Service Level ground motion records, per PEER *Guidelines* §5.5.3:

$$D + 0.5(0.8L_{0,>100psf} + 0.4L_{0,\leq 100psf}) + 1.0E$$

D = dead load, including construction dead load and superimposed dead load
$L_{0,>100psf}$ = unreduced live loads, LL, from Table 8-1 that exceed 100 psf
$L_{0,\leq 100psf}$ = unreduced live loads, LL, from Table 8-1 that are 100 psf or less
E = earthquake effects from each ground motion record pair

While not specifically referenced in the PEER load combination, effects of unbalanced earth pressures, H, have been accounted for using the following load factors:

1.0 if the effect of *H* adds to the seismic load effect,

0.6 if the effect of *H* resists the seismic load effect and *H* is permanent, or

0 if the effect of H resists the seismic load effect and *H* is not permanent

12.4. Risk-targeted Maximum Considered Earthquake (MCE_R)

The following load combination is used for nonlinear response history analysis with each pair of MCE_R ground motion records, per PEER *Guidelines* §4.2.5 and §4.2.6:

$$D + 0.5(0.8L_{0,>100psf} + 0.4L_{0,\leq 100psf}) + 1.0E$$

D = dead load, including construction dead load and superimposed dead load

$L_{0,>100psf}$ = unreduced live loads, LL, from Table 8-1 that exceed 100 psf

$L_{0,\leq 100psf}$ = unreduced live loads, LL, from Table 8-1 that are 100 psf or less

E = earthquake effects from each ground motion record pair

While not specifically referenced in PEER load combinations, effects of unbalanced earth pressures, H, have been accounted for using the following load factors:

1.0 if the effect of *H* adds to the seismic load effect,

0.6 if the effect of *H* resists the seismic load effect and *H* is permanent, or

0 if the effect of H resists the seismic load effect and *H* is not permanent

For force-controlled actions, the following load combinations used for design, per PEER *Guidelines* §6.8.3:

$$(1.2 + 0.2S_{MS})D + f_1L + 1.3I_e(Q_T - Q_{ns}) \leq \phi_s BR_n$$

$$(0.9 - 0.2S_{MS})D + 1.3I_e(Q_T - Q_{ns}) \leq \phi_s BR_n$$

S_{MS} = MCE_R spectral acceleration at short periods

D = dead load, including construction dead load and superimposed dead load

f_1 = companion load factor on live load, equal to:

= 1.0 for places of public assembly, live loads in excess of 100 psf and parking garages, or

= 0.5 for other live loads

L = reduced live loads, LL, from Table 8-1

I_e = seismic importance factor

Q_T = mean of the maximum transient force for each ground motion determined from the response history analyses

Q_{ns} = non-seismic portion of Q_T

ϕ_s = seismic resistance factor, equal to:

= □ per the relevant material design code for critical force-controlled actions,

= 0.9 for ordinary force-controlled actions, or

= 1.0 for noncritical force-controlled actions

B = factor to account for conservatism in the nominal capacity

R_n = nominal capacity per the relevant material design code

12.5. Wind Loads

<u>Allowable Stress Design (ASD):</u>

The following load combinations are used for wind Allowable Stress Design, per California Building Code §1605.3:

$$D + 0.6W + \gamma_H H$$

$$D + 0.75L + 0.75(Lr \text{ or } S \text{ or } R) + 0.45W + \gamma_H H$$

$$0.6D + 0.6W + \gamma_H H$$

<u>Load and Resistance Factor Design (LRFD):</u>

The following load combinations are used for wind Strength Design, per California Building Code §1605.2.

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W + \gamma_H H$$

$$1.2D + f_1 L + 0.5(Lr \text{ or } S \text{ or } R) + 1.0W + \gamma_H H$$

$$0.9D + 1.0W + \gamma_H H$$

D = dead load, including construction dead load and superimposed dead load

L = reduced live loads, LL, from Table 8-1

L_r = reduced roof live load

S = snow load
R = rain load
f_1 = companion load factor on live load, equal to:
= 1.0 for places of public assembly, live loads in excess of 100 psf and parking garages, or
= 0.5 for other live loads
W = wind load
γ_H = load factor on earth pressures, equal to:
= 1.0 for ASD, 1.6 for LRFD if the effect of *H* adds to the seismic load effect,
= 0.6 for ASD, 0.9 for LRFD if the effect of *H* resists the seismic load effect and *H* is permanent, or
= 0 if the effect of H resists the seismic load effect and *H* is not permanent
H = earth pressure

13 MATERIALS

13.1. Existing Building Materials

The materials specified by DeSimone in the existing building are listed in DeSimone's Construction Documents drawing S0-0.20. The material properties used for LERA's analyses are shown below. The conversion from nominal strengths to expected strengths are adopted from PEER *Guidelines* Table 4-2. The expected moduli of elasticity for concrete are computed using ACI 318-14 Eq. 19.2.2.1.b with the expected concrete strengths.

Table 13-1. Concrete properties

Nominal f'_c	Nominal E_c	Expected f'_{ce}	Expected E_{ce}
5.0 ksi	4031 ksi	6.5 ksi	4595 ksi
6.0 ksi	4415 ksi	7.8 ksi	5034 ksi
7.0 ksi	4769 ksi	9.1 ksi	5437 ksi
8.0 ksi	5098 ksi	10.4 ksi	5813 ksi
9.0 ksi	5407 ksi	11.7 ksi	6165 ksi
10.0 ksi	5700 ksi	13.0 ksi	6499 ksi

For the concrete in the mat foundation slab, a statistical analysis of the concrete test reports that were provided as part of the original construction record was performed per ACI 214.4R-03. The results of this study are summarized in Table 13-2. See Appendix B.1.1 for the full statistical analysis data. The equivalent in-place concrete strength has been used for review of the mat capacity.

Table 13-2. Mat concrete properties

Specified f'_c	Mean Strength $\dot{f}c$	Equivalent to Specified Strength f'_{ce}	Expected E_{ce}
6.0 ksi	8.77 ksi	7.71 ksi	5005 ksi

Table 13-3. Reinforcement properties

Standard	Nominal f_y	Expected f_{ye}	Expected f_u
ASTM A706 Grade 60	60 ksi	69 ksi	95 ksi
ASTM A615 Grade 75	75 ksi	82 ksi	114 ksi

Table 13-4. Post-tensioning strand properties

Standard	Nominal f_{pu}
ASTM A416	270 ksi

13.2. Retrofit Materials

The material properties for the retrofit piles are shown below. The conversion from nominal strengths to expected strengths are adopted from PEER *Guidelines* Table 4-2. The expected moduli of elasticity for concrete are computed using ACI 318-14 Eq. 19.2.2.1.b with the expected concrete strengths.

Table 13-5. Minimum pile grout properties

Nominal f'_c	Nominal E_c	Expected f'_{ce}	Expected E_{ce}
5.0 ksi	4031 ksi	6.5 ksi	4595 ksi

Table 13-6. Pile steel casing properties

Standard	Nominal f_y	Nominal f_u
Casing	80 ksi	100 ksi

14 TESTING PROGRAM

Testing shall be undertaken to verify the pile installation method and expected load carrying capacity for the production piles. We propose accomplishing a pile load test for the displacement pile in the basement as shown in Figure 14-1:

- Propose to reuse mat connection test hole:

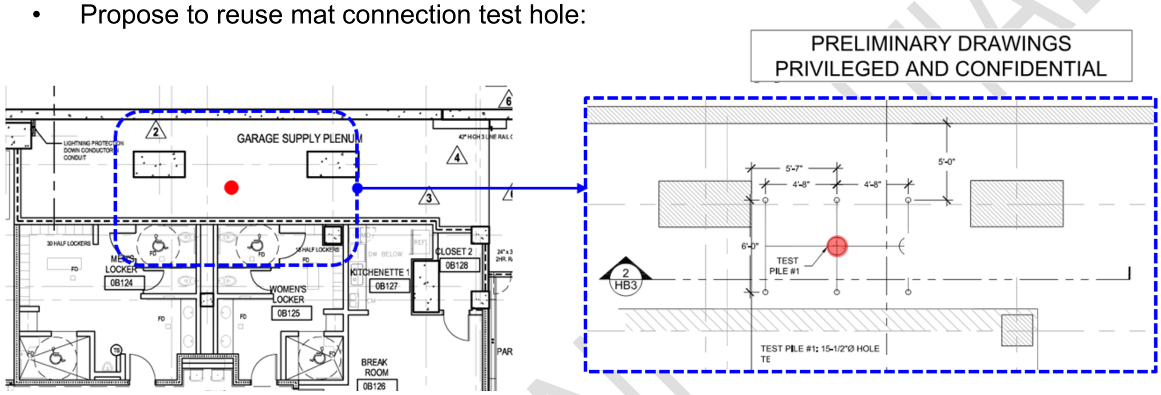


Figure 14-1. New Displacement Pile Load Test Location

During construction every pile will be load tested to validate its load carrying capacity. This is a valuable benefit of the displacement pile installation technique.

Load testing of the proposed final connection between the retrofit piles and existing tower mat in the tower basement was completed this summer as part of the original Test Pile Program to validate the load carrying capacity of the design. The test successfully demonstrated that the connection can support loads in excess of the proposed 2000k ultimate pile design load. More information about this test is included in Appendix C.8.

15 TILT CORRECTION OPTION

The retrofit scheme has an add alternate option to achieve some amount of tilt correction…

APPENDIX A: LETTER TO CITY OF SAN FRANCISCO DBI REGARDING LERA EVALUATION OF THE EXISTING STRUCTURE

APPENDIX B: REVIEW OF EXISTING CONDITIONS

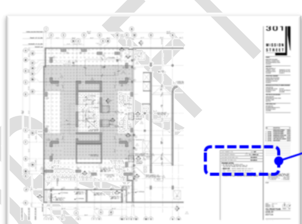
B.1. Tower Mat

A review of the project record showed that the tower mat, with the exception of the lower 11' section of mat in the elevator pit area, was cast overnight in one continuous pour on May 17, 2005. Concrete strength testing that was accomplished on cylinders collected from this pour were reviewed and the in-place concrete strength was calculated using ACI 214. Additionally, a review of thermal record from the mat pour shows that a tight temperature differential, well within the recommended limits included in CTL's report, was achieved between the concrete in the middle of the mat and that close to the surface. This supports the use of the calculated in-place concrete strength for calculations related to the existing mat capacity.

B.1.1. In-place Concrete Strength

The original design drawings required the mat concrete strength, f'c = 6,000 psi. Per a statistical analysis of the mat concrete cores per ACI 214.4R-03, the equivalent in-place concrete strength of the mat slab may be taken as: f'c = 7,710 psi. LERA has used this value in our calculations. A summary of the statistical analysis results is shown below. See Appendix C.3 for the full statistical analysis data and cylinder test results.

Specified Concrete Strength = 6,000 psi



Original Mat Design Drawing S2-0.B1.11 by DCE

SLAB THICKNESS U.O.N.:	t = 120"
SLAB CONCRETE STRENGTH:	f'c = 6000 psi
SLAB BASIC BOTTOM BARS U.O.N.:	S2-0.B1.12
SLAB BASIC TOP BARS U.O.N.:	S2-0.B1.13

IN-PLACE CONCRETE STRENGTH IS ≈ 28% GREATER THAN SPECIFED

Actual In-Place Concrete Strength = 7,710psi
(based on statistical analysis of concrete test results, per ACI guidelines)

Example Mat Concrete Test Report

CYL #	TEST AGE	TEST DATE	CURE TYPE	DIMENSIONS (in) DIAMETER X HEIGHT	AREA (sq. in)	MAXIMUM LOAD (lb)	CORR FACT	STRENGTH (psi)
004350G	91	09/15/2006	L	4.00 X 8.00	12.57	112,631	1.00	8960
004350H	91	09/15/2006	L	4.00 X 8.00	12.57	113,507	1.00	9030

B.1.2. Mat Construction Thermal Record[AJK10]

B.1.2.1. CTL Thermal Control Plan (Selected Sections)

February 22, 2006

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Greg Scott
Webcor Concrete
31145 San Antonio Street
Hayward, CA 94544

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Fax: 510-476-3001
gregs@webcor.com

Thermal Modeling and Thermal Control Plan for Control of Temperatures and Temperature Differences in the 10-ft thick Foundation Placements at 301 Mission, San Francisco, CA
CTLGroup Project No. 313088

Dear Greg:

As authorized, CTLGroup has performed thermal modeling and developed this thermal control plan for the 10-ft thick elevator pit and main foundation placements of the referenced project.

This plan was developed utilizing a Cemex concrete mix design that was developed specifically to minimize mass concrete issues in several massive foundations placements for the Bay Bridge. Its use on the 10-ft thick foundations of this project reduces the efforts needed to address mass concrete concerns.

The plan was also developed around the same performance-based approach to prevention of thermal cracking that is being used on various sections of the Bay Bridge and the Benicia-Martinez Bridge (and other structures across the US). In this performance-based approach, the temperature difference limit is tailored to the properties of the concrete and structure. The resulting temperature difference limit is a function of the in-place strength of the concrete. As the strength increases, the concrete is more tolerant of thermal stresses and is therefore more resistant to thermal cracking. The use of this approach greatly reduces the likelihood of early age thermal cracking, does not compromise the durability of the concrete, and reduces the time of construction.

The following provides a summary of the major points of this thermal control plan. Detailed information is presented in later sections of this plan.

A. **Concrete Mix Design**
The foundations will utilize Cemex Concrete Mix No. 32039. If a different concrete is utilized, this thermal control plan will need to be revised.

B. **Concrete Placement**
The means and methods of the concrete placement are left to the discretion of Webcor. The simultaneous use of several concrete pump trucks (or equivalent) is recommended.

C. **Methods of Controlling Temperature Differentials**
Surface insulation is required to keep the temperature difference within acceptable limits. This insulation will completely cover all exposed concrete and formwork, and must be installed so that its ability to protect the concrete is not compromised by wind or rain. This insulation will remain in place until the foundation concrete has cooled adequately. In general, this will take several weeks to occur. Insulation can be temporarily removed to facilitate work, if performed using procedures described in this plan.

Main Office 5400 Old Orchard Road Skokie, Illinois 60077-1030 Phone 847-965-7500 Fax 847-965-6541
Northeast Office 5565 Sterrett Place, Suite 312 Columbia, Maryland 21044-2685 Phone 410-997-0400 Fax 410-997-8480

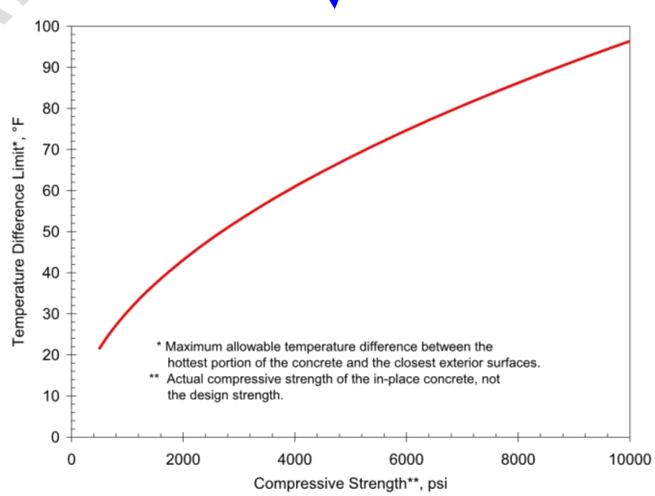


Figure 1 – Temperature Difference Limit for the Foundation Placements

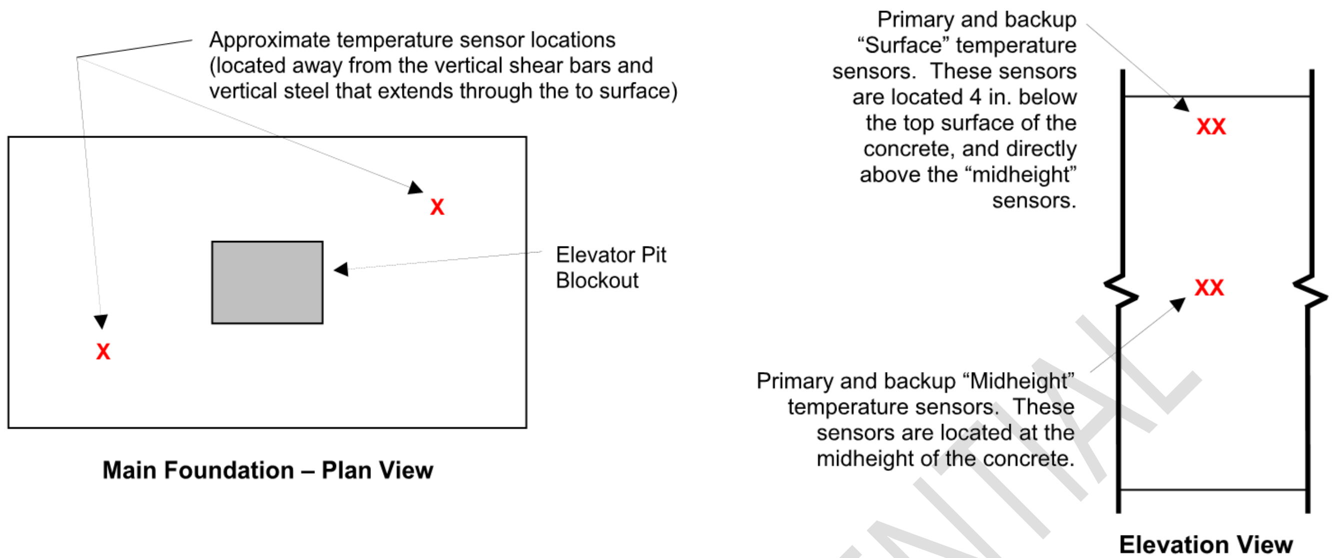
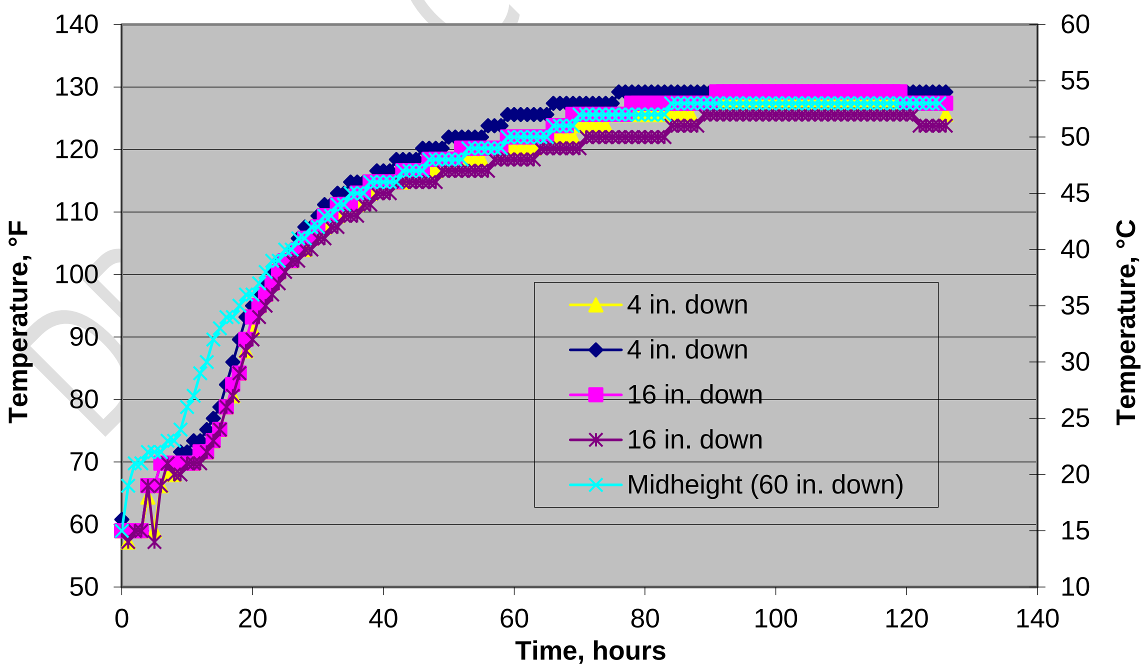


Figure 2 – Temperature Sensor Locations (Not to Scale)

B.1.2.2. Thermal Monitoring During Concrete Placement

The following is chart from the project record for the thermal monitoring of the mat during concrete placement:



Thermal Monitoring Record – 5/17/2006 – 5/23/2006

B.2. Existing Precast Concrete Piles

B.2.1. Pile Dowel Reinforcement

The precast concrete piles were designed by a subcontractor (Kie-Con) based on the design requirements provided by DeSimone in the contract documents. The original design of these piles called for 8 #9 dowels to be cast into the mat slab. 4 of the dowels were cast into the piles and 4 were to be grouted into the piles in the field. The full precast pile shop drawing submittal package is provided in Appendix C.5 for reference.

Through LERA's review of the project record, it was determined that some of the dowels were omitted to alleviate congestion with mat rebar. For 'non-tension' piles, it was proposed that the 4 dowels to be grouted in the field be omitted entirely, leaving only 4 #9 dowel bars. For 'tension piles', a design with 6 #9 dowel bars was proposed. See Figure A-1 for the original and revised dowel designs. The proposed changes were ultimately accepted by DeSimone in their response to RFI 238 (Figure A-2). DeSimone identified the tension piles in their response to RFI 212R1 (Figure A-3).

Construction photos taken prior to mat concrete placement further confirm that the dowels were omitted. See Figure A-4.

Additionally, some piles were driven too low, requiring a cast-in-place build up at the top of the pile to engage with the mat.

Based on the responses to RFI's 196, 196R1, 212, 212R1, and 238, LERA has determined that there were 4 basic pile types constructed:

- No Build-Up Required: Tension Pile (6 #9 dowels | 8 strands cast into mat)
- No Build-Up Required: Non-Tension Pile (4 #9 dowels | 8 strands cast into mat)
- Build-Up Required: Tension Pile (8 #9 dowels | No strands cast into mat)
- Build-Up Required: Non-Tension Pile (4 #9 dowels | No strands cast into mat)

A full mapping of the different pile types based on the RFI responses is provided in Figure A-5. LERA's analysis has accounted for the reduced dowels at the top of the piles.

The full set of RFIs related to the pile dowel reinforcement are provided in Appendix C.6 for reference.

Original Pile Dowel Design: **Field Revised Design:**

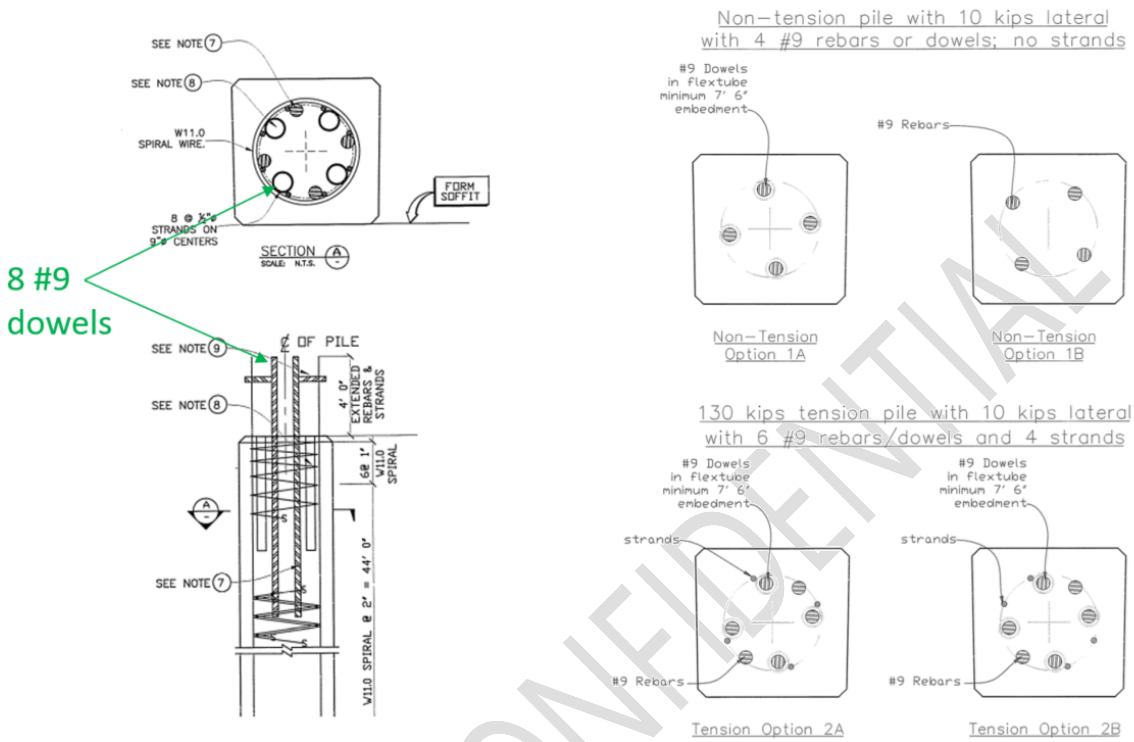
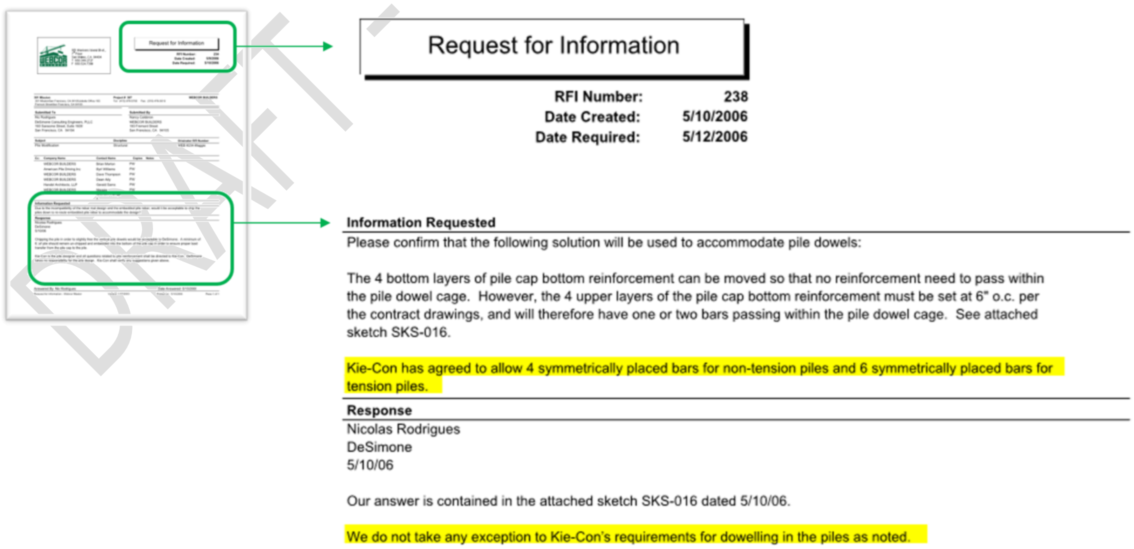


Figure A-1. Excerpts from Pile Shop Drawing and Revisions Proposed in RFI-212R1

Pile Dowels into Mat Slab:



Request for Information

RFI Number:	238
Date Created:	5/10/2006
Date Required:	5/12/2006

Information Requested

Please confirm that the following solution will be used to accommodate pile dowels:

The 4 bottom layers of pile cap bottom reinforcement can be moved so that no reinforcement need to pass within the pile dowel cage. However, the 4 upper layers of the pile cap bottom reinforcement must be set at 6" o.c. per the contract drawings, and will therefore have one or two bars passing within the pile dowel cage. See attached sketch SKS-016.

Kie-Con has agreed to allow 4 symmetrically placed bars for non-tension piles and 6 symmetrically placed bars for tension piles.

Response

Nicolas Rodrigues
DeSimone
5/10/06

Our answer is contained in the attached sketch SKS-016 dated 5/10/06.

We do not take any exception to Kie-Con's requirements for dowelling in the piles as noted.

Figure A-2. Excerpt from DeSimone Response to RFI-238

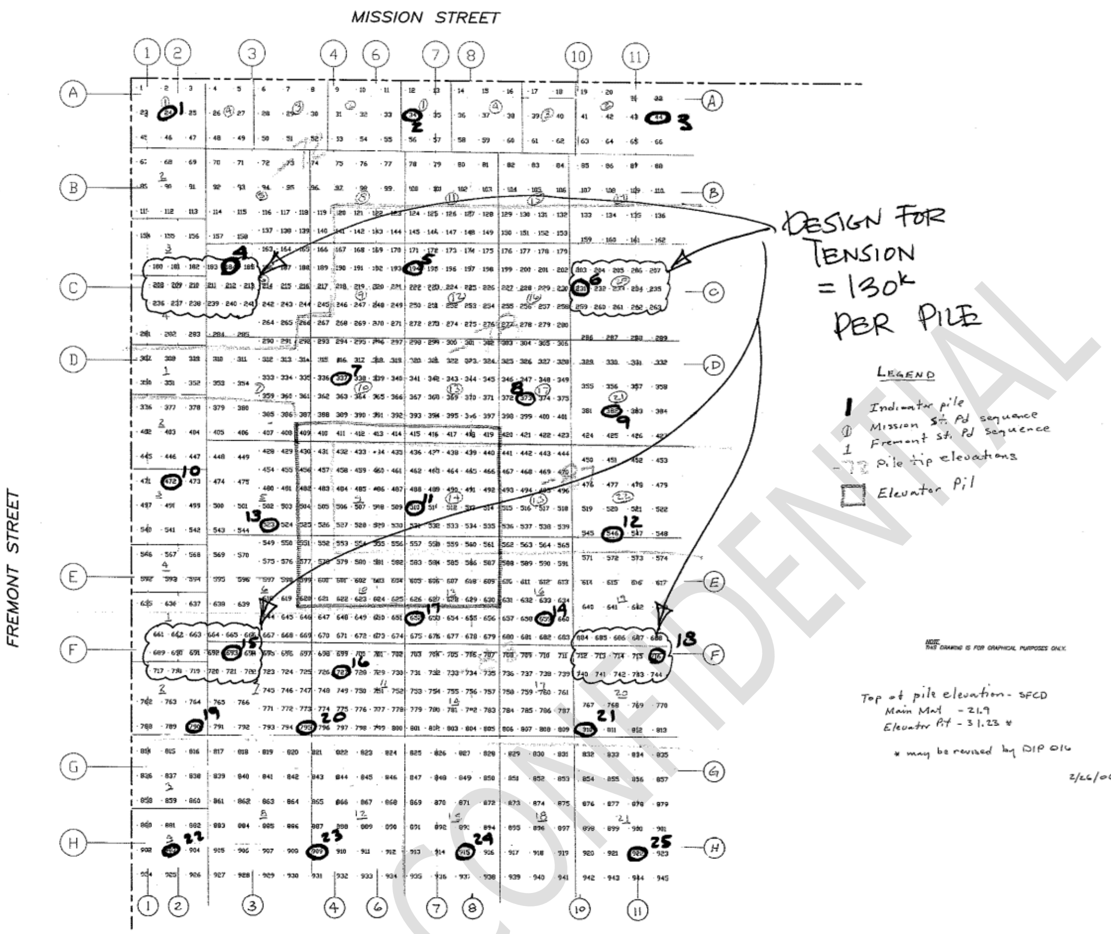
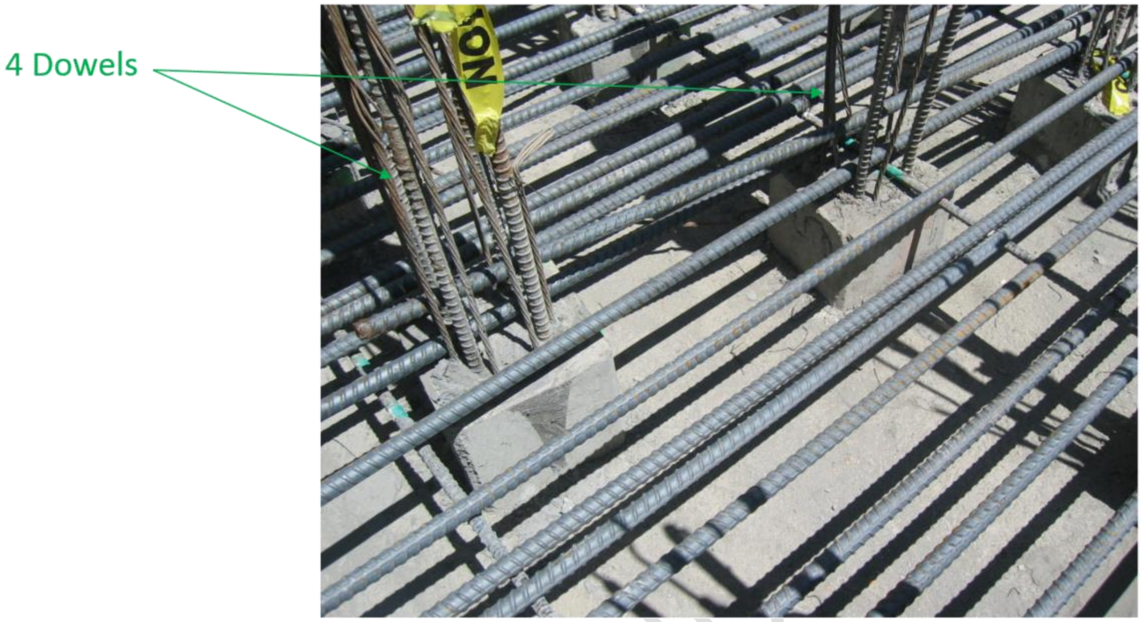


Figure A-3. DeSimone Identification of Tension Piles in Response to RFI-212R1



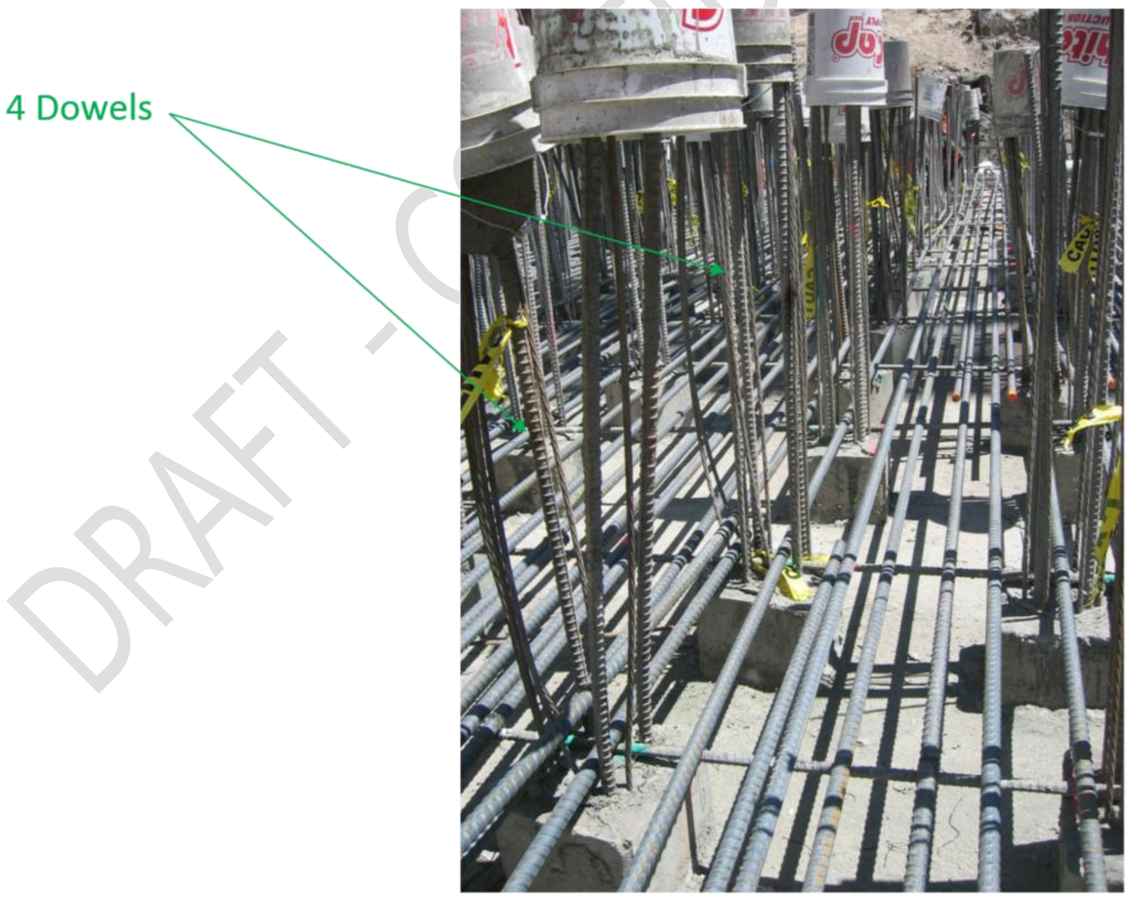


Figure A-4. Construction Photos showing four dowels at the top of piles

Tension Pile - No build up [6 #9 Bars | 8 Strands | 14"x14"]	65	6.9%
Tension Pile - Build up req'd [8 #9 Bars | No Strands | 16"x16"]	1	0.1%
Compression Pile - No build up [4 #9 Bars | 8 Strands | 14"x14"]	856	90.4%
Compression Pile - Builld up req'd [4 #9 Bars | No Strands | 16"x16"]	16	1.7%
Broken Pile	9	1.0%

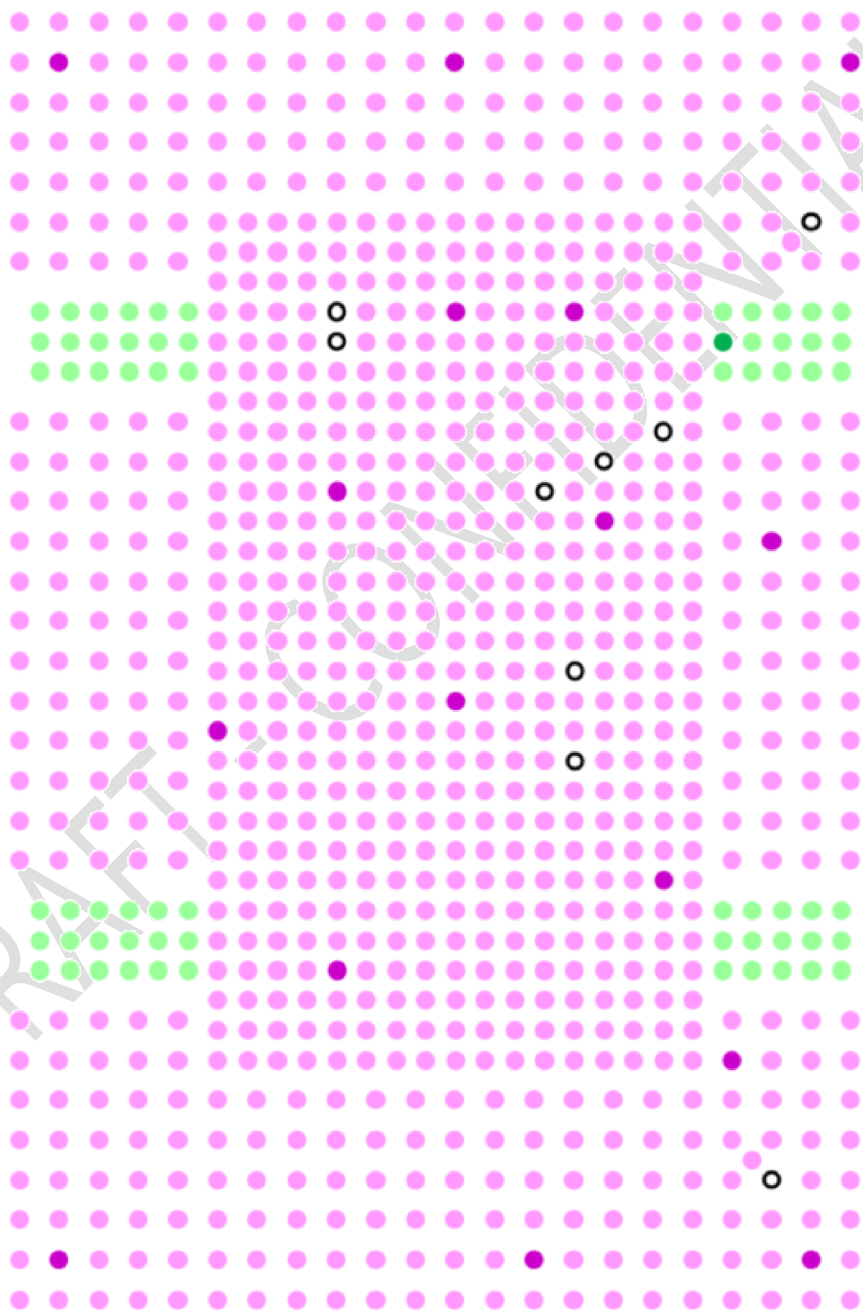


Figure A-5. LERA Mapping of Existing Pile Dowel Reinforcement

B.2.2. Pile Cut-off Length

Following the indicator pile program, Treadwell & Rollo issued a document titled "Summary of Indicator Pile Driving and Production Pile Recommendations". As titled, this document provided recommendations for the production pile driving operation. Specifically, the document gave the required resistance needed to achieve the design pile strength.

Because of the variability of the sand bearing layer, it was not known precisely how deep each pile would need to be driven to achieve this resistance. To accommodate this unknown, an allowance for cut off was provided, should the pile achieve the required resistance at a shallower elevation than anticipated.

The precast concrete pile shop drawing submittal showed an allowance for 10 feet of cut off. This was revised to 12 feet in RFI-047R1.

Per the record of pile driving activities (provided in full in Appendix C.7), 86 piles met refusal early, requiring cut off greater than 12 feet. This non-conformance was reviewed and accepted by T&R and DeSimone in RFI-162 (see Figure A-7), while noting that the resulting reduction in lateral capacity was acceptable. A mapping of the piles requiring cut off greater than 12 feet is provided in Figure A-8.

For these piles, the dowel bars will not be fully developed for flexure at the interface with the bottom of the mat. LERA's analysis has accounted for this as-built conditions in two ways:

1) Lateral bending capacities are reduced where the dowel bars are not fully developed.
2) The LPILE analyses described in Section 7.6.2.2 are modified to account for the as-built length of the upper reinforcement zone.

Pile Cutoff Lengths:

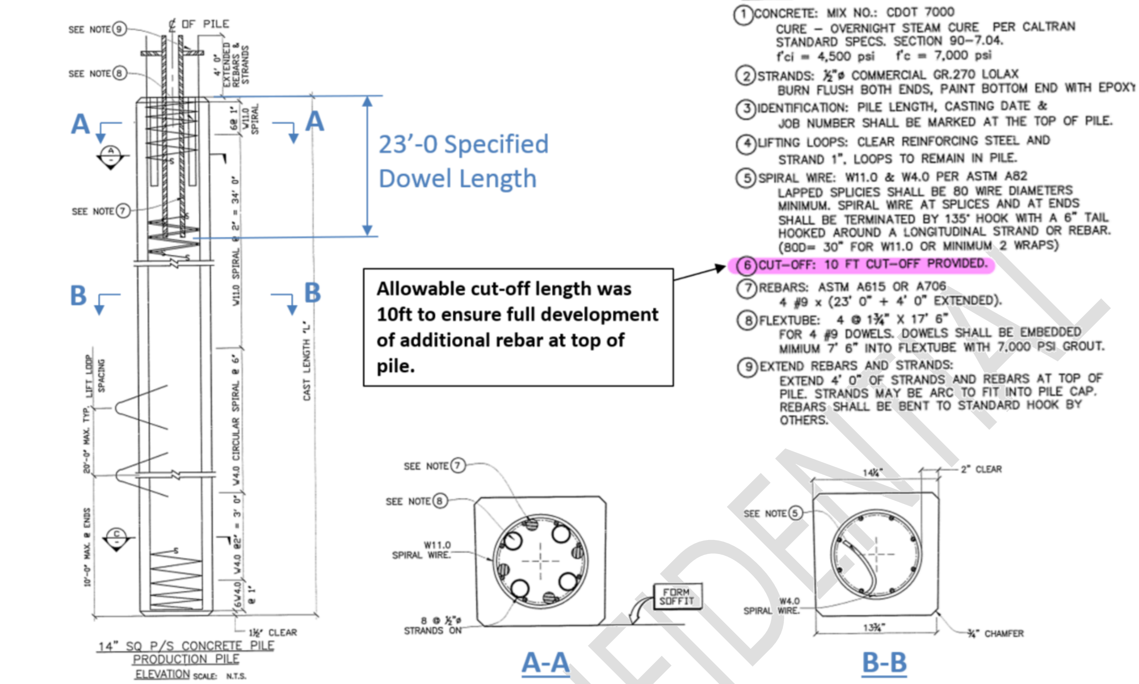
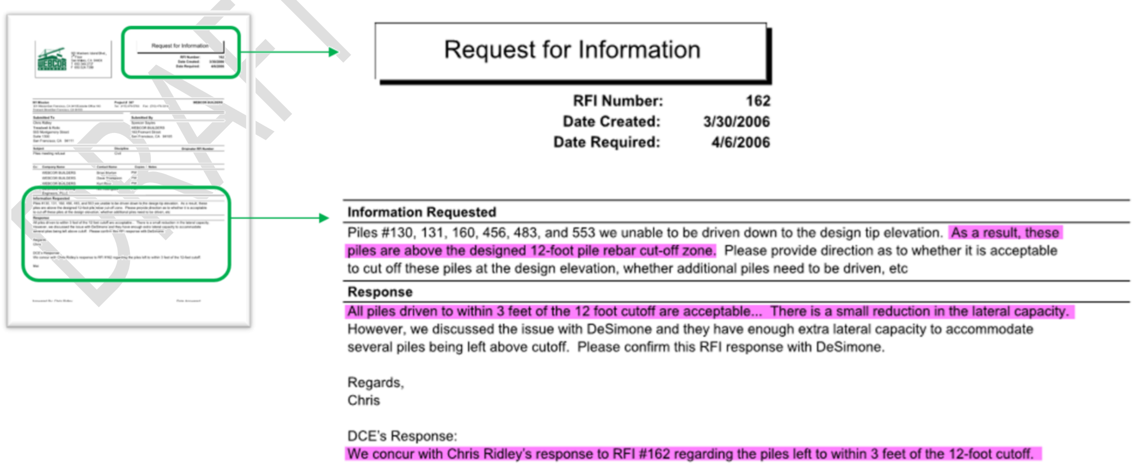


Figure A-6. Excerpt from Pile Shop Drawings Showing Allowable Pile Cut-Off Length

Pile Cutoff Lengths:



Request for Information

RFI Number:	162
Date Created:	3/30/2006
Date Required:	4/6/2006

Information Requested

Piles #130, 131, 160, 456, 483, and 553 we unable to be driven down to the design tip elevation. As a result, these piles are above the designed 12-foot pile rebar cut-off zone. Please provide direction as to whether it is acceptable to cut off these piles at the design elevation, whether additional piles need to be driven, etc

Response

All piles driven to within 3 feet of the 12 foot cutoff are acceptable... There is a small reduction in the lateral capacity. However, we discussed the issue with DeSimone and they have enough extra lateral capacity to accommodate several piles being left above cutoff. Please confirm this RFI response with DeSimone.

Regards,
Chris

DCE's Response:
We concur with Chris Ridley's response to RFI #162 regarding the piles left to within 3 feet of the 12-foot cutoff.

Figure A-7. Excerpt from T&R and DeSimone Response to RFI-162

> 12ft less than design tip elevation	42	4.4%
>15ft less than design tip elevation	44	4.6%
Broken Pile	9	1.0%

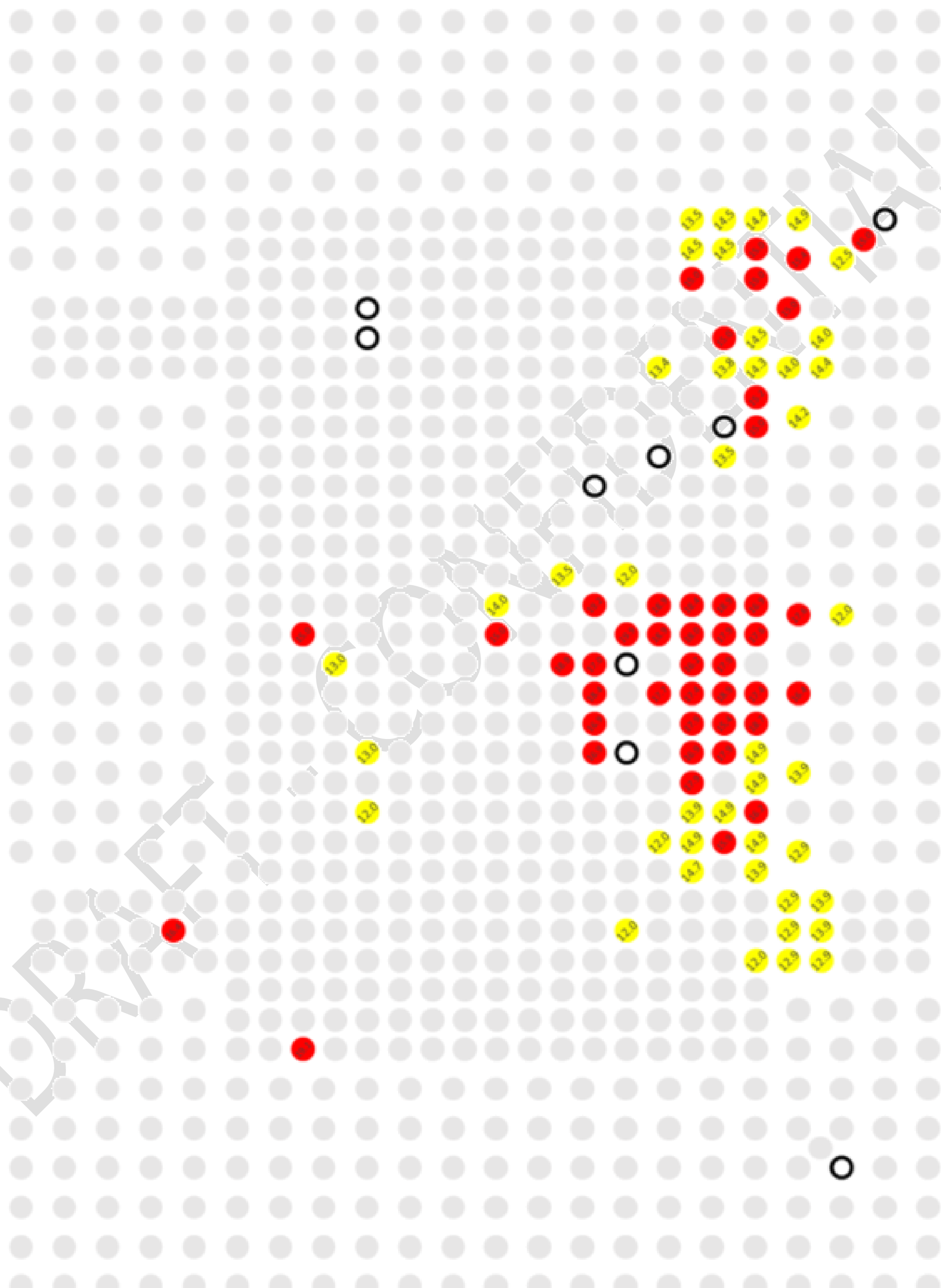


Figure A-8. LERA Mapping of Existing Piles with Cut-Off Exceeding 12 Feet

B.2.3. Pile As-Built Locations

It is LERA's understanding that a comprehensive as-built pile survey was not completed during the foundation construction. However, per project RFI-202, several piles in the SW corner were surveyed. Per this survey, it was observed that the average pile was mislocated by approximately 10" from its theoretical position. The worst case piles are 30" from the theoretical position. See Figure A-9 for the survey data provided in RFI-202. Figure A-10 shows the surveyed pile positions overlaid with the theoretical locations.

Request for Information

RFI Number: 202
Date Created: 4/24/2006
Date Required: 4/24/2006

- Average > 10" off theoretical locations
- Worst piles are 30" off theoretical locations

PILE AS-BUILT SURVEY

Pile Number				
635	0.03	North of Plan Location	1.17	East of Plan Location
661	0.15	North of Plan Location	0.13	West of Plan Location
662	0.21	North of Plan Location	1.02	West of Plan Location
663	0.50	South of Plan Location	0.05	East of Plan Location
664	0.23	South of Plan Location	0.04	East of Plan Location
689	0.44	North of Plan Location	0.07	West of Plan Location
690	0.32	South of Plan Location	0.52	West of Plan Location
691	0.15	South of Plan Location	0.54	West of Plan Location
717	1.12	South of Plan Location	2.22	West of Plan Location
718	0.22	South of Plan Location	2.48	West of Plan Location
719	0.28	North of Plan Location	0.33	West of Plan Location
720	0.37	South of Plan Location	1.06	West of Plan Location
762	0.82	North of Plan Location	0.89	East of Plan Location
763	0.21	North of Plan Location	0.80	West of Plan Location
764	0.42	South of Plan Location	0.09	West of Plan Location
788	0.44	North of Plan Location	0.37	East of Plan Location

Dimensions and elevations are in decimal feet.

Figure A-9. Excerpt from RFI-202

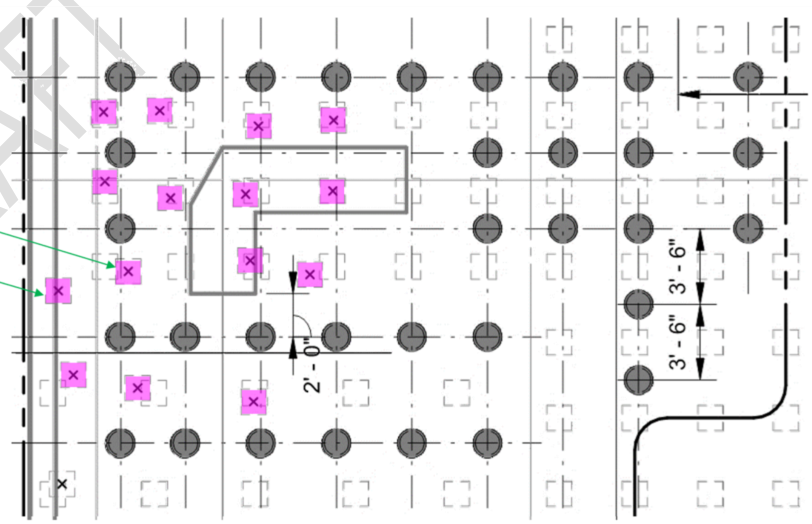
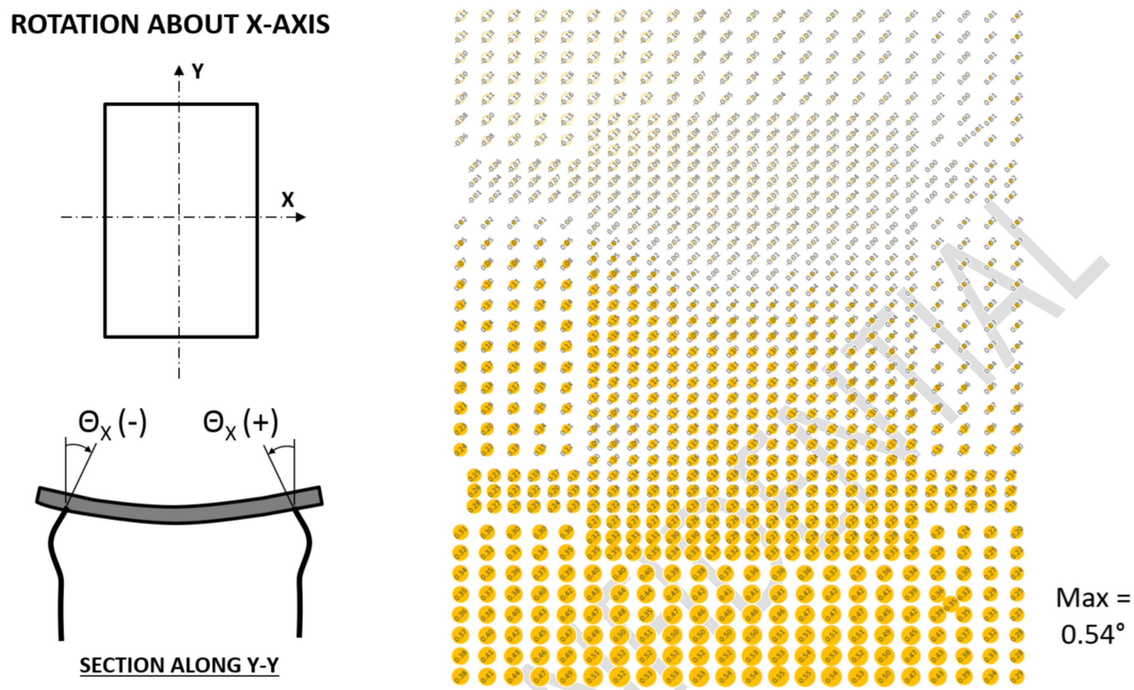
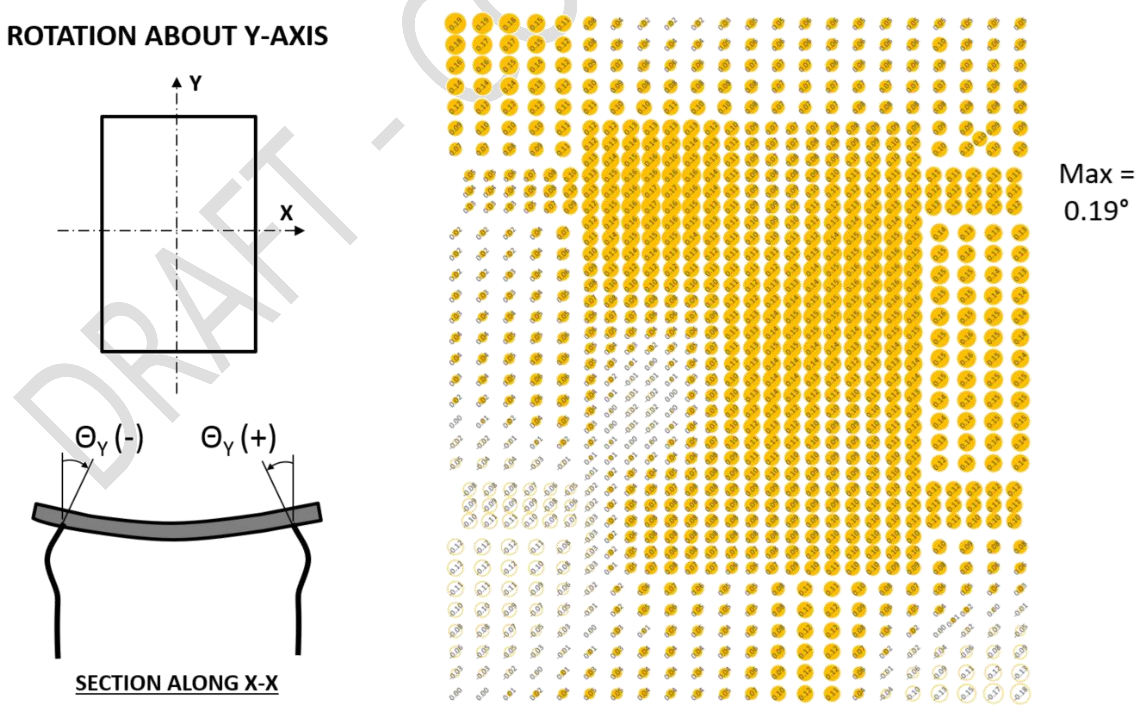


Figure A-10. Overlay of Surveyed and Theoretical Pile Locations

B.2.4. Pile Head Rotations Due to Settlement[AJK11]





B.3. Soil Mix Shoring Wall Between Tower and Mid-Rise Structure[AJK12]

APPENDIX C: REFERENCE DOCUMENTS

C.1. Original Building Structural Drawings

C.1.1. Original Building Structural Drawing List

The following list of original building structural drawings by DeSimone Consulting Engineering were referenced by LERA in designing the tower foundation retrofit. Copies of a few key reference drawings are included in Section C.1.2.

Drawing No	Drawing Title	Rev	Date
S-0	**General Information**		
S0-0.10	DRAWING INDEX PG 1	-	8/30/2007
S0-0.11	DRAWING INDEX PG 2	-	4/4/2008
S0-0.15	STRUCTURAL DESIGN CRITERIA	1	5/31/2006
S0-0.16	STRUCTURAL REVIEW LETTERS	2	5/31/2006
S0-0.20	GENERAL NOTES	4	11/1/2006
S-2.0	**Full Project Plans**		
S-2.0.B5.11	LEVEL B5 PLAN	6	4/11/2007
S2-0.B4.11	LEVEL B4 PLAN	4	3/16/2007
S2-0.B3.11	LEVEL B3 PLAN	4	3/16/2007
S2-0.B2.11	LEVEL B2 PLAN	4	3/16/2007
S2-0.B1.01	LEVEL B1 - REFERENCE PLAN	-	3/9/2005
S2-0.B1.11	LEVEL B1 WEST PLAN	6	5/31/2006
S2-0.B1.12	LEVEL B1 WEST PLAN BOTTOM REINFORCEMENT	4	3/15/2006
S2-0.B1.13	LEVEL B1 WEST PLAN TOP REINFORCEMENT	5	8/18/2006
S2-0.B1.14	LEVEL B1 WEST PLAN PILE LOCATION	4	3/15/2006
S2-0.B1.21	LEVEL B1 EAST PLAN	5	3/16/2007
S2-0.01.01	LEVEL 1 - REFERENCE PLAN	-	9/18/2005
S2-0.01.11	LEVEL 1 WEST PLAN	8	8/16/2006
S2-0.01.21	LEVEL 1 EAST PLAN	6	10/10/2007
S2-0.02.01	LEVEL 2 - REFERENCE PLAN	-	11/18/2005
S2-0.02.11	LEVEL 2 WEST PLAN	7	8/18/2006

Drawing No	Drawing Title	Rev	Date
S2-0.02.12	LEVEL 2 WEST PLAN, PORTE COCHERE STRUCTURE	4	1/16/2008
S2-0.02.21	LEVEL 2 EAST PLAN	6	1/16/2008
S2-0.03.01	LEVEL 3 - REFERENCE PLAN	-	11/18/2008
S2-0.03.11	LEVEL 3 WEST PLAN	5	8/18/2006
S2-0.03.21	LEVEL 3 EAST PLAN	4	10/10/2007
S2-1	**TOWER FRAMING PLANS**		
S2-1.04. 01	LEVELS 4-7, 14-16, 23, 24 PLAN	4	7/31/2006
S2-1.08.01	LEVELS 8, 12 , 17, 21 PLAN	3	8/30/2006
S2-1.09. 01	LEVELS 9-11 & 18-20 PLAN	4	8/30/2006
S2-1.13.01	LEVELS 13, 22 PLAN	3	8/30/2006
S2-1.25.01	LEVEL 25 PLAN	3	8/30/2006
S2-1.26.01	LEVEL 26 PLAN	4	3/16/2007
S2-1.27.01	LEVEL 27 PLAN	3	8/30/2006
S2-1.28.01	LEVELS 28-41 PLAN	2	8/30/2006
S2-1.42.01	LEVELS 42, 45 PLAN	3	8/30/2006
S2-1.43.01	LEVELS 43,44 PLAN	3	8/30/2006
S2-1.46.01	LEVEL 46 PLAN	3	8/30/2006
S2-1.47.01	LEVEL 47 PLAN	3	8/30/2006
S2-1.48.01	LEVELS 48-56 PLAN	3	8/30/2006
S2-1.54.01	LEVELS 54-56 PLAN	2	8/30/2006
S2-1.57.01	LEVEL 57 PLAN	3	8/13/2007
S2-1.58.01	LEVELS 58 PLAN	4	8/13/2007
S2-1.59.01	LEVEL 59 PLAN	6	8/13/2007
S2-1.60.01	LEVEL 60 PLAN	5	8/13/2007
S2-1.61.01	LEVEL 61 PLAN	4	8/13/2007
S2-1.62.01	LEVEL 62 PLAN	2	8/13/2007
S2-3	**MID-RISE FRAMING PLANS**		
S2-3.04.01	LEVELS 4-9 PLANS	3	10/10/2007
S2-3.04.01	LEVEL 10 PLAN	-	10/10/2007
S2-3.11.01	LEVEL 11 PLAN	-	10/10/2007
S2-3.12.01	LEVEL 12 PLAN	4	1/23/2008
S2-3.13.01	LEVEL 13 PLAN	2	1/23/2008
S2-3.14.01	LEVEL 14 PLAN	2	1/23/2008

Drawing No	Drawing Title	Rev	Date
S3-1	**FOUNDATION**		
S3-1.01	TYPICAL FOUNDATION SECTIONS AND DETAILS	5	3/16/2007
S3-1.11	FOUNDATION SECTIONS AND DETAILS	4	11/1/2006
S3-1.12	FOUNDATION SECTIONS AND DETAILS	3	11/1/2006
S3-1.13	FOUNDATION SECTIONS AND DETAILS	2	11/1/2006
S3-1.14	FOUNDATION SECTIONS AND DETAILS	4	7/19/2006
S3-1.15	FOUNDATION SECTIONS AND DETAILS	3	1/18/2007
S3-2	**VERTICAL SYSTEMS**		
S3-2.01	TOWER COLUMN SCHEDULE	5	4/16/2007
S3-2.02	TOWER COLUMN DETAILS	4	5/31/2006
S3-2.03	TOWER COLUMN DETAILS	-	4/16/2007
S3-2.08	PODIUM/OFFICE/AMENITIES COLUMS SCHEDULE	7	10/10/2007
S3-2.11	TOWER SHEAR WALL ELEVATIONS	4	4/16/2007
S3-2.12	TOWER SHEAR WALL ELEVATIONS	6	1/23/2008
S3-2.21	PODIUM SHEAR WALL SCHEDULE	4	5/31/2006
S3-2.23	TOWER LINK BEAM DETAILS AND SCHEDULE	5	4/16/2007
S3-2.24	MID-RISE LINK BEAM SCHEDULE AND DETAILS	3	1/23/2008
S3-2.31	TOWER SHEAR WALL PLANS	3	12/30/2005
S3-2.32	TOWER SHEAR WALL PLANS	3	12/30/2005
S3-2.33	TOWER SHEAR WALL PLANS	2	11/18/2005
S3-2.34	TOWER SHEAR WALL PLANS	1	11/18/2005
S3-2.35	TOWER SHEAR WALL PLANS	1	11/18/2005
S3-2.36	TOWER SHEAR WALL PLANS	1	11/18/2005
S3-2.37	TOWER SHEAR WALL PLANS	-	11/18/2005
S3-2.38	TOWER SHEAR WALL PLANS	-	11/18/2005
S3-2.39	TOWER SHEAR WALL PLANS	1	4/16/2007
S3-2.41	TOWER SHEAR WALL PLANS	2	5/31/2006
S3-2.42	TOWER SHEAR WALL PLANS	4	1/23/2008
S3-2.43	TOWER SHEAR WALL PLANS	3	6/19/2007
S3-2.44	TOWER SHEAR WALL ELEVATIONS	-	1/23/2008
S3-2.51	TOWER SHEAR WALL DETAILS	3	12/30/2005
S3-2.52	TOWER SHEAR WALL DETAILS	2	12/30/2005
S3-2.53	TOWER SHEAR WALL DETAILS	1	11/18/2005

Drawing No	Drawing Title	Rev	Date
S3-2.54	TOWER SHEAR WALL DETAILS	2	4/16/2007
S3-2.55	TOWER SHEAR WALL DETAILS	-	11/18/2005
S3-2.56	TOWER SHEAR WALL DETAILS	-	1/18/2005
S3-2.57	TOWER SHEAR WALL DETAILS	1	5/31/2006
S3-2.58	TOWER SHEAR WALL DETAILS	4	3/3/2006
S3-2.59	TOWER SHEAR WALL DETAILS	2	5/31/2006
S3-2.60	TOWER SHEAR WALL DETAILS	1	8/13/2007
S3-2.71	TOWER MOMENT FRAME SCHEDULE AND DETAILS	3	4/16/2007
S3-2.72	TOWER MOMENT FRAME DETAILS	4	5/31/2006
S3-2.73	TOWER MOMENT FRAME DETAILS	2	5/31/2006
S3-3	**SUPERSTURCTURE**		
S3-3.01	TYPICAL CONCRETE DETAILS	3	7/19/2006
S3-3.02	TYPICAL CONCRETE DETAILS	3	10/10/2007
S3-3.03	TYPICAL CONCRETE DETAILS	1	11/18/2005
S3-3.04	TYPICAL CONCRETE DETAILS	2	10/10/2007
S3-3.05	CONCRETE DETAILS	2	6/19/2007
S3-3.06	SUPERSTURCTURE SECTIONS AND DETAILS	-	10/10/2007
S3-3.07	SUPERSTURCTURE SECTIONS AND DETAILS	-	10/10/2007
S3-3.11	SUPERSTURCTURE SECTIONS AND DETAILS	8	1/23/2008
S3-3.12	SUPERSTURCTURE SECTIONS AND DETAILS	5	10/10/2007
S3-3.13	SUPERSTURCTURE SECTIONS AND DETAILS	6	4/4/2008
S3-3.14	SUPERSTURCTURE SECTIONS AND DETAILS	3	10/10/2007
S3-3.15	SUPERSTURCTURE SECTIONS AND DETAILS	3	10/10/2007
S3-3.16	SUPERSTURCTURE SECTIONS AND DETAILS	3	1/16/2008
S3-3.17	SUPERSTURCTURE SECTIONS AND DETAILS	3	1/16/2008
S3-3.18	SUPERSTURCTURE SECTIONS AND DETAILS	6	10/10/2007
S3-3.19	SECTIONS AND DETAILS	2	8/16/2007
S3-3.20	SUPERSTURCTURE SECTIONS AND DETAILS	2	1/16/2008
S3-3.21	TYPICAL POST-TENSION SECTIONS AND DETAILS	-	11/18/2005
S3-3.22	TYPICAL POST-TENSION SECTIONS AND DETAILS	1	5/31/2006
S3-3.23	TYPICAL CMU SECTIONS AND DETAILS	-	7/31/2006
S3-3.31	TYPICAL STEEL SECTIONS AND DETAILS	1	8/13/2007
S3-3.32	TOWER ROOF DETAILS	2	8/13/2007

Drawing No	Drawing Title	Rev	Date
S3-3.33	MID-RISE ROOF SECTIONS AND DETAILS	-	12/13/2006
S3-3.34	TOWER ROOF DETAILS	-	8/13/2006

C.1.2. Selected Original Building Structural Drawings

C.2. Tower Mat Reinforcement Shop Drawings

C.3. In-Place Mat Concrete Strength

C.3.1. Statistical Analysis of Concrete Strength Test Results

Per ACI 214.4R-03

C.3.2. Cylinder Test Results

C.4. Thermal Control Plan During Mat Concrete Placement

C.5. Precast Concrete Pile Shop Drawings

C.6. Pile Dowel Reinforcement RFIs

C.7. Pile Driving Record

C.8. Mat Connection Test Results

Testing was accomplished in the garage plenum in the NE corner of the tower mat to verify that the proposed final connection between the piles and tower mat meets the required ultimate design capacity of 2000k. See Figure C- for the test location. The tested connection consisted of ¼” weld beads at 6” on center around the outside of the outer casing within the depth of the mat (see Figure C-).

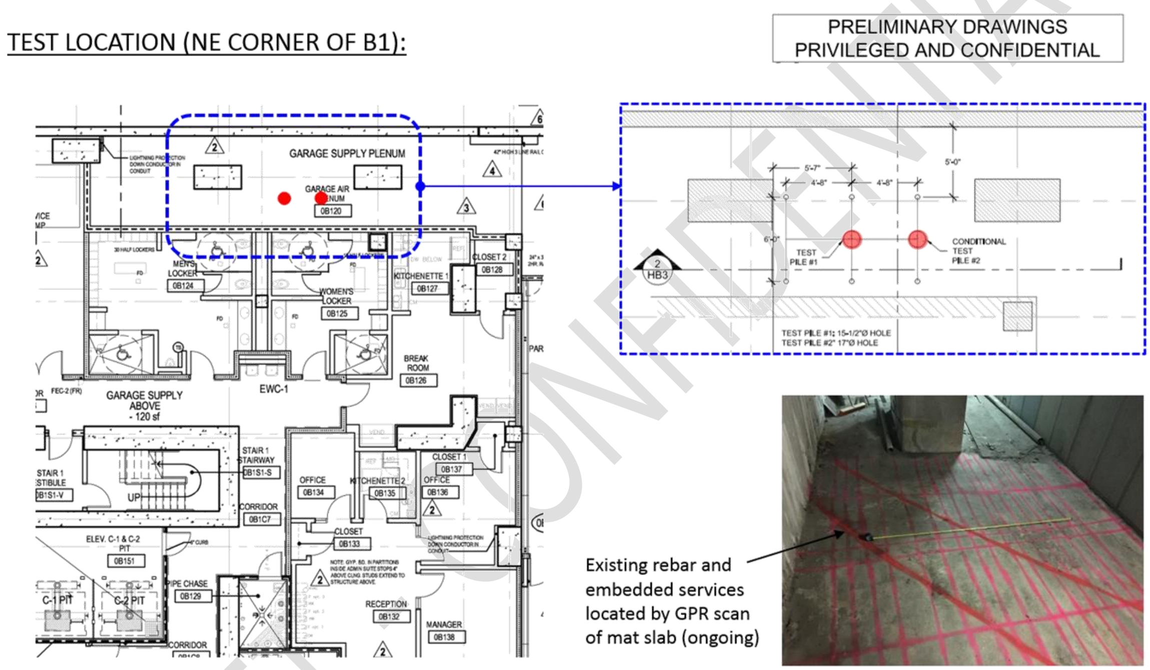


Figure C-_. Mat Connection Test Location

INTERIOR TEST #1 SETUP

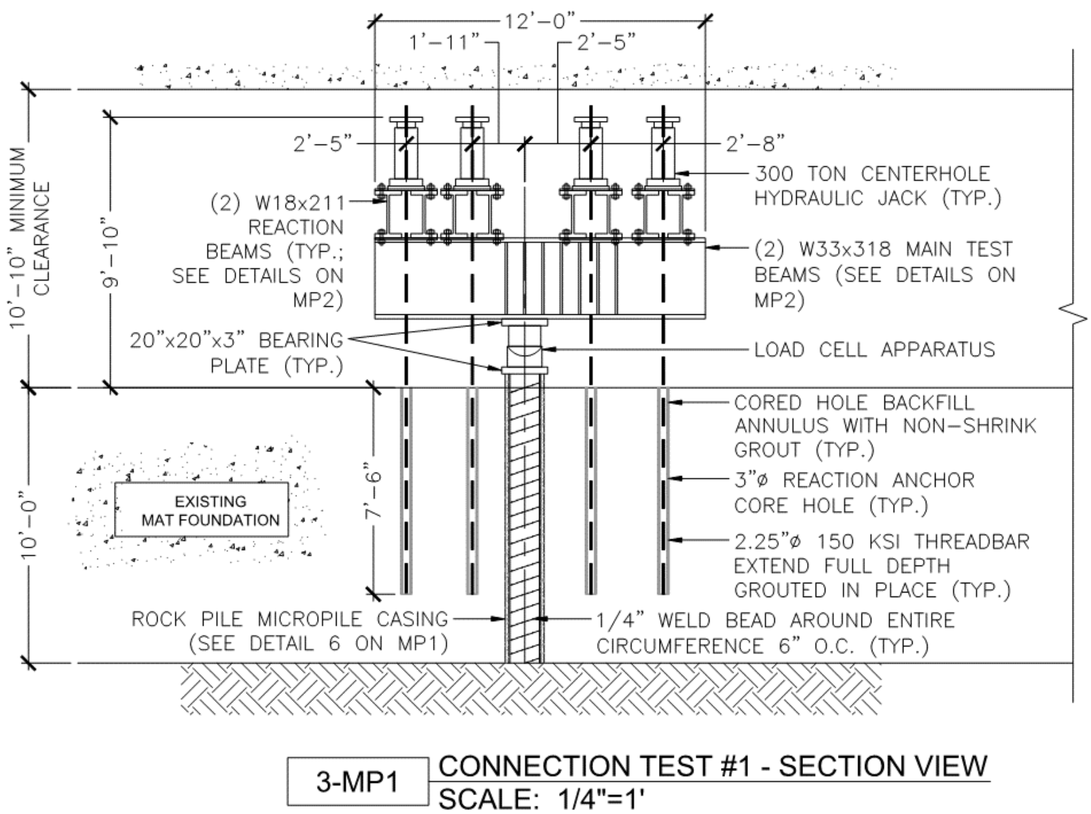


Figure C-_. Mat Connection Test Section