DIGGING IN FOR A 34-STORY NOVEL HIGHRISE TOWER WITH CONSTRUCTION PHASING TAKING CENTER STAGE

Saiful Islam, Ph.D., SE
Chairman & CEO, Saiful Bouquet
Los Angeles, CA

Martin B. Hudson, Ph.D., GE
Adjunct Professor, UCLA; Principal Geotechnical Engineer, Hudson Geotechnics, Inc.
Los Angeles, CA

Shafiq Ibrahim
Associate Principal, Saiful Bouquet
Los Angeles, CA

Dong-Won Kim, Ph.D., PE
Project Manager – Advanced Technology, Saiful Bouquet
Los Angeles, CA

Rishabh Singhvi, PE
Project Manager, Saiful Bouquet
Los Angeles, CA

Kenneth S. Hudson, Ph.D. RG, EIT
Principal Geoscientist, Hudson Geotechnics, Inc.
El Segundo, CA

Abstract

The Landmark Apartment building is a 34-story, 349-foot-tall residential tower constructed in West Los Angeles. The high-rise uses concrete core walls as lateral force resisting system designed using a Performance Based Design Approach. The new tower was constructed through the existing post-tensioned long-span four-level subterranean parking structure within the podium of an existing high-rise structure. Considerable challenges included undertaking seismic design while mitigating adverse impacts on the existing high rise and podium structure that were to remain fully operational during construction, and to coordinate the design of new construction with multiple shoring and enabling construction phases.

The design utilized the provisions of the Los Angeles Tall Buildings Structural Design Criteria, requiring peer review, including impacts on the existing structure. The ground motions developed for the design of the structure were relatively large due to the proximity of the site to the Santa Monica Fault. The foundation system for the new high-rise consisted of

a large pile cap foundation supported on drilled shafts for limiting the influence due to settlement of the new high-rise on the existing structures.

Multiple stages were evaluated for maintaining structural stability throughout construction. Existing footings had to be strengthened at certain locations to support new shear walls supporting the podium for unbalanced soil pressures in the temporary condition. The existing structure was also carefully studied to avoid triggering the need for additional strengthening for the existing basement walls due to updated earth bearing pressures per the latest building code.

The new pile cap foundation was split into two parts to allow space for the raker shoring deadmen footings. Differential settlements were studied to limit differential movement between new and existing foundations.

The complexity of building a new highrise within an existing basement had numerous difficulties that were overcome with creative and detailed engineering and new techniques.

Introduction

Landmark Los Angeles is the first residential high-rise to be built west of the 405 Freeway in more than 40 years. The modern 34-story, 415,000 sf concrete tower features 376 units and is built over 4 levels subterranean parking. The new shimmering glass-and-steel structure rises 349 feet in height with floor-to- ceiling windows, as shown in Figure 1. The new tower also includes a Jr. Olympic size pool and an amenities deck, and repurposing the former surface parking lot on top of the subterranean parking into a one-acre, landscaped park. Designed with features inspired by a forest, the green space is privately maintained and offers a meadow and children's play fountain, seating areas, paved walkways, and a grassy lawn to enjoy outdoor concerts and performances.

Figure 1 Completed Tower

The Landmark tower resides within an existing 1980's full city block development. The existing development included a 3 to 4 levels of post-tensioned subterranean parking covering the entire site with a 16-story steel office tower and a 2-story steel supermarket rising from the podium. The new tower, which replaced the existing 2-story market that had been supported on top of the subterranean parking levels, was built through the existing post-tensioned long-span subterranean parking structure, as shown in Figure 2. A schematic of the property before and after construction of the new tower is shown in Figure 3. A plan of the site is shown in Figure 4..

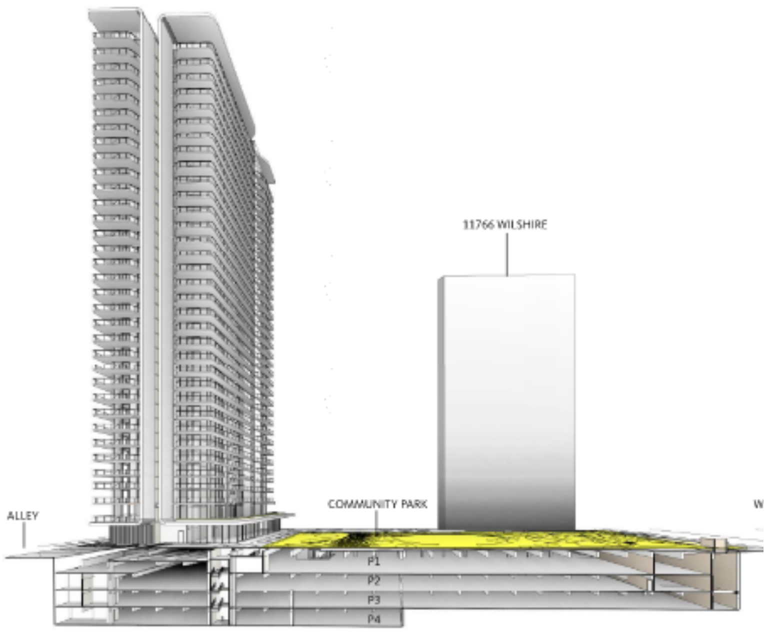

Figure 2 Development Vision

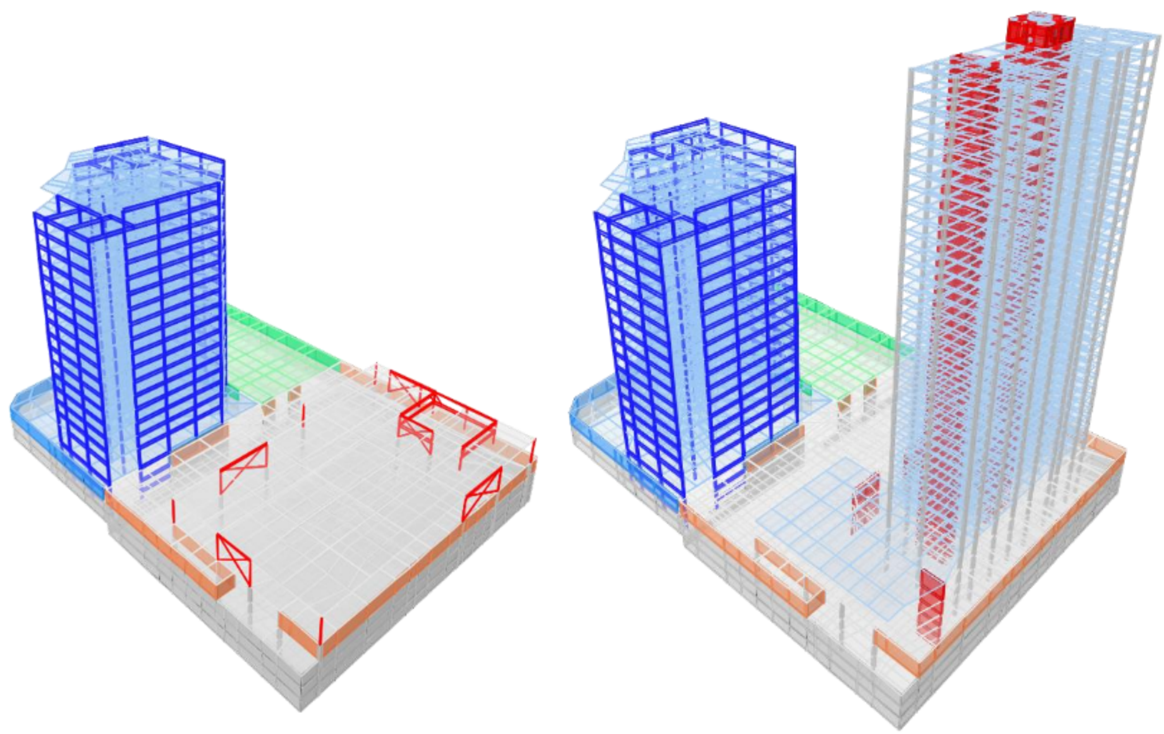
Figure 3 Schematic of Structure a) Before New Tower and b) After New Tower

In addition to the new tower, the project design also involved evaluating the entire development as one structure with both new and existing towers rising from the common podium, strengthening of the subterranean parking where required, and strengthening long-span PT deck/girders supporting the garden. The new tower consists of concrete construction with core walls as the lateral resisting system and post-tensioned slabs. The seismic design of the tower followed a Performance Based Design approach.

The project required significant modification of the existing building, including demolition of the portion of the subterranean levels where the new tower was constructed (i.e. creating of a “glory-hole” the size of the footprint of the new tower), which in-turn required temporary shoring of the existing basement walls in proximity to the demolished portion of the basement. A plan of the site showing some of the major structural elements is shown in Figure 4.

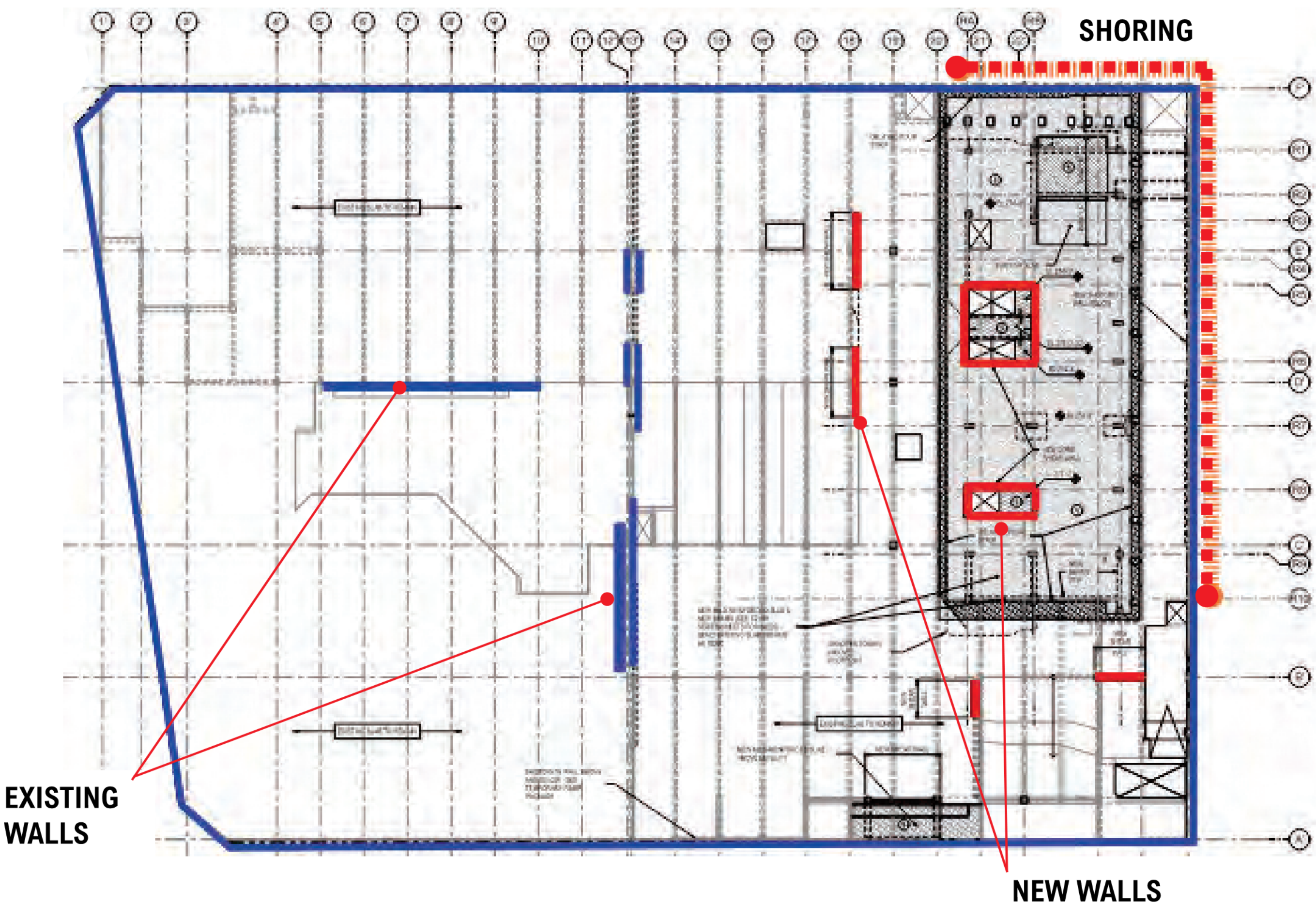

Figure 4 Plan View with Major Structural Elements at Basement Level

Threading a new tower through an existing long-span post-tensioned subterranean garage is extremely complicated by itself but when combined with the other project constraints, it truly needed every bit of design creativity and teamwork to succeed.

Structural System

The typical floors of the high-rise tower are comprised of two-way post-tensioned concrete slabs supported by concrete columns. The podium and below grade slabs are two-way mild-reinforced slabs and beams. The lateral force resisting system is comprised of two specially reinforced concrete cores that were designed based on the Los Angeles Tall Buildings Structural Design Criteria. All new tower construction was supported by six feet thick pile cap foundation supported on drilled shafts for limiting the differential movement of the new high-rise and the resulting influence on the existing construction.

To limit the impact on the existing high-rise, any additional diaphragm forces imparted from the new high-rise to the existing structural system were resisted by additional shear walls introduced below grade and by strengthening existing foundations. The challenge of keeping the existing high-rise and the majority of the below grade parking operational during construction of the new high-rise required additional structural members to be introduced to maintain stability at each stage of construction.

Structural Challenges

The successful implementation of the project required the design team to work around some very big project constraints and be extremely creative in addressing some of the structural challenges that came with inserting a 34-story new tower into an existing city-block project. The client had a firm direction for the design team that the existing tower and majority of the below grade parking structure supporting the existing tower shall remain fully operational throughout the construction of the new high-rise. Additionally, any work required on the existing tower as a result of the addition of the new tower would likely make the development infeasible. Accordingly, multiple structural and geotechnical studies were undertaken to ensure no impact to the existing structure in the final condition (after the high-rise is fully constructed), as well as no impact during all intermediate enabling and demo conditions as well.

Structural analysis was performed for each major construction stage. The existing and new structural and foundation elements were evaluated for each phase as the existing high-rise and portion of the parking structure had to remain fully operational and accessible to the occupants:

1. Existing Stage - To establish baseline for existing structure

2. Enabling Phase – To study the performance of the structure with additional shear walls installed. These shear walls are installed to allow for the next phase which involved demo of large portions of the existing diaphragms to make way for the new high-rise tower. To ensure the parking structure remained operational throughout the project, the team had to reframe the parking entrances on the west side, requiring the introduction of additional walls to stabilize the existing structure, as shown in Figure 5.

3. Demo Phase – This phase focused on redistribution of diaphragm forces including the soil retaining force as the portion of the existing slabs were removed to make way for the new high-rise.

4. Final Condition – Once the new tower is fully constructed and stitched to the existing diaphragms, the existing structural systems were evaluated for additional stresses due to the new tower verifying that there is no adverse impact on the existing high-rise due to the new construction.

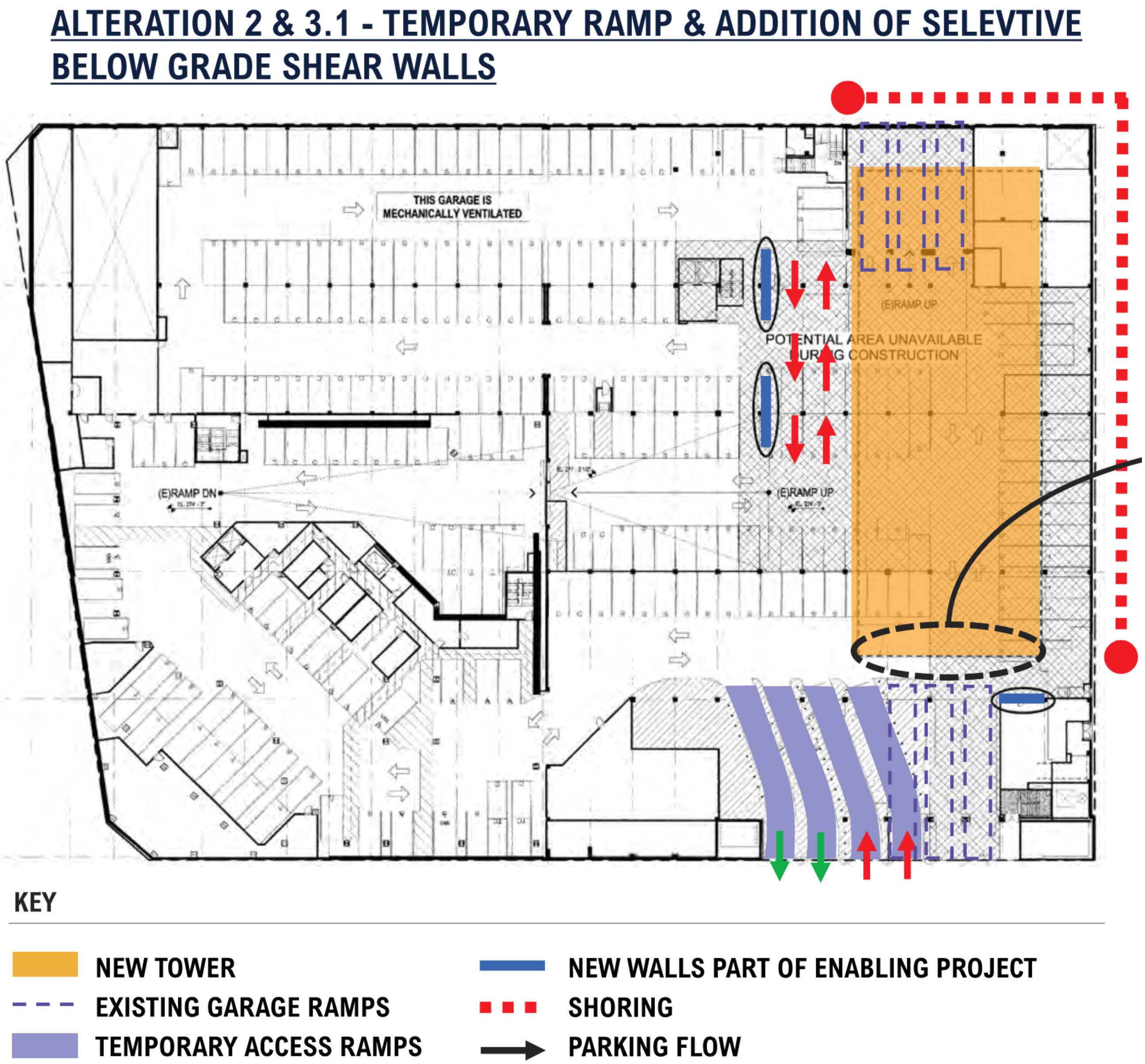

Figure 5 Enabling Ramps and Shoring in Plan View

Design Criteria

The structure was designed using a Performance Based Design approach in accordance with the Los Angeles Tall Building Structural Design Criteria. In accordance with the design criteria, a peer review was performed; since the new high-rise structure was in proximity to the pre-existing high-rise structure on the site, consideration in the peer review had to be made as to the seismic influence of the new structure on the existing high-rise. The existing high-rise structure was continuously occupied by tenants throughout the construction of the adjacent high-rise structure, and therefore it was desired to minimize structural modifications to the existing structure except at the location of the new high-rise.

Geotechnical Conditions

In 2017, a geotechnical investigation of the site was conducted by Amec Foster Wheeler (Amec Foster Wheeler 2017), who had also performed (under one of its predecessor company names) the original geotechnical investigation in the 1980's for the existing structure. The 1980's investigation included exploratory borings and the 2017 investigation included Cone Penetration Tests (CPTs), at the locations shown in Figure 6. A geologic cross-section through the site (taken at the location shown on Figure 6) is provided in Figure 7. The natural soils beneath the site encountered during the investigations are alluvial deposits consisting of clay, silt, silty sand, and sand. Varying amounts of gravel were encountered in the sand deposits. The natural soils were firm throughout the depth explored. The groundwater levels were measured as part of the 2017 investigation at depths of 42 and 47 feet below the existing grade adjacent to the building at CPT-4 and CPT-5, respectively. The measured groundwater depths correspond to the elevation of the gravel subdrain layer which had been originally installed below the slab-on-grade just above the foundation level. Groundwater was encountered in the 1980's borings at depths of 33 to 44 feet bgs, prior to the permanent lowering of the groundwater by the subdrain dewatering system installed during construction of the original development. As part of construction of the existing structure, a permanent dewatering system was installed to depress the groundwater to just below the basement slab-on-grade. The dewatering system includes a layer of gravel with perforated pipes and sumps. Therefore, since the time of the original construction of the building, the groundwater level at the site has been constrained to the gravel dewatering layer just beneath the slab. During construction of the current project, additional temporary dewatering was performed to further depress the groundwater level to allow for installation of new pile caps.

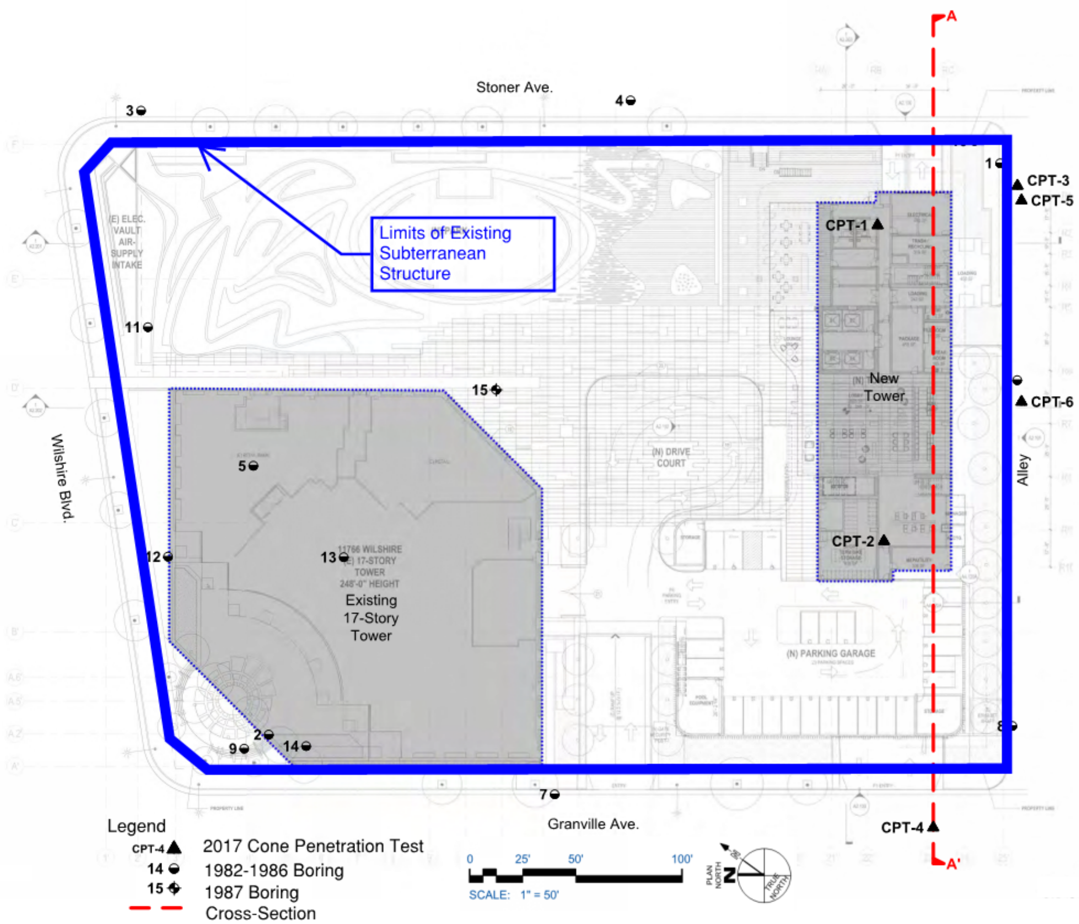

Figure 6 Site Plan

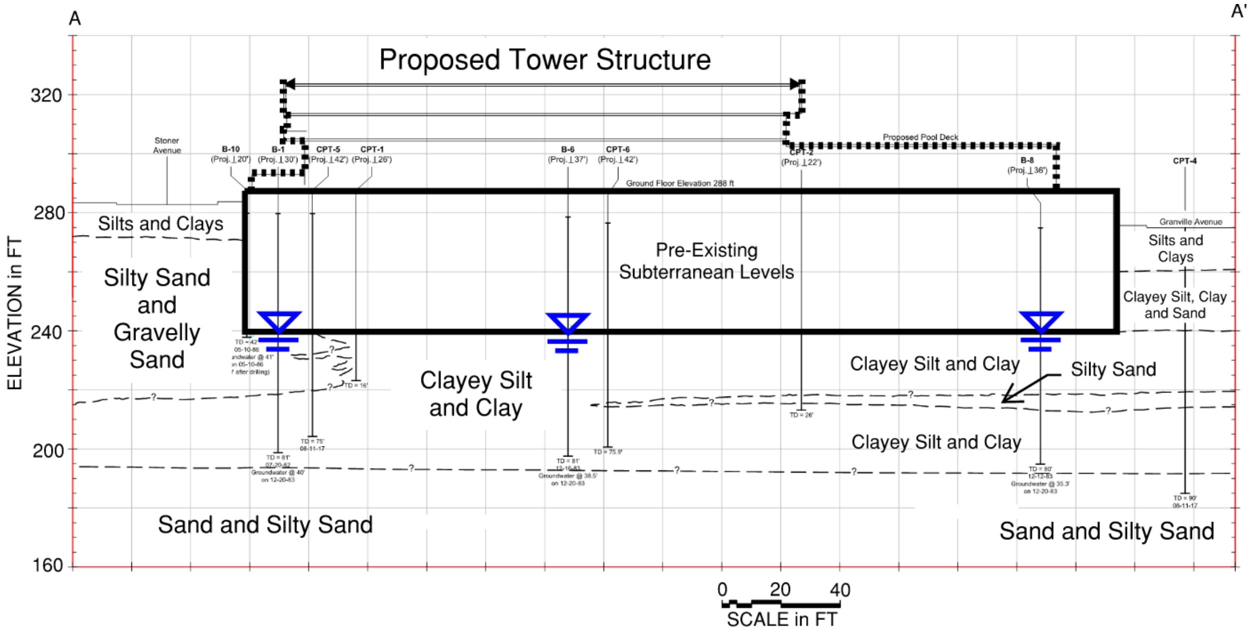

Figure 7 Cross-Section A-A'

Ground Motions

To characterize the seismic hazard at the site, a site-specific ground motion hazard analysis was performed in accordance with Section 21 of ASCE 7-16 and LATBSDC guidelines (American Society of Civil Engineers 2017; Los Angeles Tall Buildings Structural Design Council 2017) and ground motions were selected and modified in accordance with Section 16.2 of ASCE 7-16.

The V_{S30} (time-averaged shear wave velocity in the upper 30 m) was taken as the time-averaged V_S in the 30 m below the bottom of the building foundation: 431 m/s. The basin depth terms, $Z_{1.0}$ and $Z_{2.5}$ (the depth to shear wave velocity of 1.0 and 2.5 km/s, respectively) were taken as 0.178 and 2.5 km, respectively, from the California Department of Transportation (Caltrans) ARS Online tool (Caltrans 2017).

The weighted average of four ergodic ground motion models (GMMs) was taken as the total response for the site. The GMMs are from the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 project (Abrahamson et al. 2014; Boore et al. 2014; Campbell and Bozorgnia 2014; Chiou and Youngs 2014). Equal weights of 0.25 are applied to the four GMMs used. Idriss (2014) was not included because the V_{S30} is less than 450 m/s, which is outside of the range of applicability of that model. The GMMs provide ergodic median component ground motion (RotD50) estimates as a function of V_{S30}, $Z_{1.0}$ and $Z_{2.5}$. These are the weighting schemes and GMMs recommended by the national seismic hazard model project (NSHMP) (Petersen et al. 2024).

A probabilistic seismic hazard analysis (PSHA) was performed using the computer program EZ-FRISK (Risk Engineering 2015) to develop a uniform hazard spectrum for the ground motion with a 2% probability of being exceeded in 50 years. A deterministic seismic hazard analysis (DSHA) was also performed in EZ-FRISK considering all the nearby faults contributing significant seismic hazard to the site. The maximum of the 84th percentile spectral ordinates from the DSHA were used to assemble a componsite deterministic response spectrum.

The computed 5%-damped median ground motions were converted to maximum response direction ground motions (RotD100) using the scaling factors recommended in the Shahi-Baker study (Shahi and Baker 2014). The resulting probabilistic response spectrum in the maximum response direction was considered as the fault normal (FN) spectrum for the project and the geomean was considered as the fault parallel (FP).

In accordance with Chapter 21 of ASCE 7-16, the probabilistic risk-targeted response spectrum (MCE_R) is taken as S_a in the direction of maximum horizontal response represented by a 5% damped acceleration response spectrum that is expected to achieve a 1% probability of collapse within a 50-year period (Section 21.2.1.1 of ASCE 7-16). The risk-targeted coefficients, C_{RS} and C_{R1}, were taken from Figures 22-18 and 22-19 in ASCE 7-16. The C_{RS} value was applied for periods less than or equal to 0.2 second, values were linearly interpolated between C_{RS} and C_{R1} between 0.2 and 1 second, and the C_{R1} value was applied for periods greater than 1 second.

In accordance with Section 21.2.2 of ASCE 7-16, the deterministic MCE response spectrum was taken as the largest of the maximum direction response spectrum of the compiled 84th percentile deterministic events and the deterministic lower limit at each spectral period. In accordance with Chapter 21 of ASCE 7-16, the site-specific MCE_R was taken as the minimum of the deterministic MCE with lower limit considered and probabilistic MCE_R response spectra at each spectral period. The resulting MCE_R is presented in Figure 8.

The PSHA was also used to develop the level of ground motion recommended in the LATBSDC procedure (Los Angeles Tall Buildings Structural Design Council 2017) for the serviceability evaluation. The probability of exceedance for the serviceability ground motions is a 50% probability of exceedance in 30 years (designated as the Service Level Earthquake, SLE), for 2.5% of critical structural damping. To develop the site-specific SLE spectrum, factors to adjust the probabilistic spectrum from 5% to 2.5% damping were developed using the model developed by Rezaeian et al. (2014). The 2.5% damped SLE spectrum is presented in Figure 8.

Two Conditional Mean Spectra (CMS) were developed according to Section 16.2.1.2 of ASCE 7-16 and Baker (2011). The periods selected for the two CMS were conditioned on the 1^{st} and 2^{nd} fundamental periods of the structure: 6 and 1 second, respectively. The period range of interest used to determine the 75% MCE_R minimum was selected as 0.2 second and 9.0 seconds to include the elastic modes necessary to achieve 90% mass participation in both horizontal directions and 1.5 times the maximum fundamental period of the structure. The CMS were developed for both the FN and FP directions.

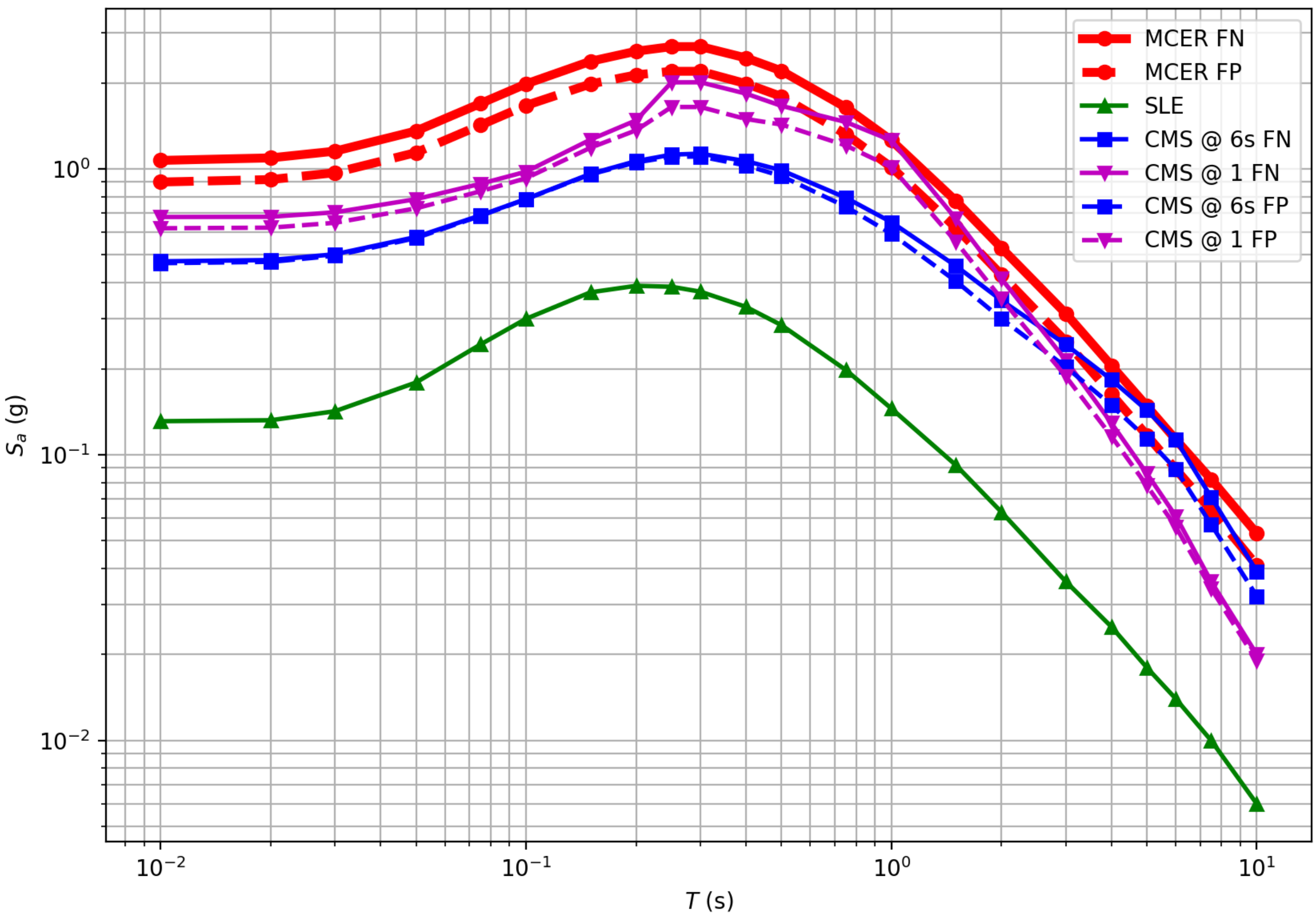

Figure 8. Site-specific Response Spectra.

A suite of eleven acceleration-time series records pairs (consisting of two orthogonal horizontal ground motion components) were developed for each CMS target spectrum, resulting in two suites of eleven horizontal pairs for each suite. Each record pair was rotated to FN and FP directions. Deaggregations within the period range of interest were computed to determine suite characteristics for the selection of seed time series. The time series were selected from the Next-Generation Attenuation (NGA) West2 database (Ancheta et al. 2014). Records were selected to minimize the mean squared error (MSE) between the record's response spectrum and the target spectrum within the period range of interest when a scale factor (SF) between 0.2 and 4.0 is applied (i.e. records were not selected where a scale factor outside of that range was necessary). Moment magnitude (**M**) (Hanks and Kanamori 1979), Joyner-Boore distance (R_{jb}) (Joyner and Boore 1981), fault mechanism, recording site conditions (V_{S30} and site class), highest useable frequency (f_{HU}), significant duraction ($D_{5\text{-}95}$) (Kempton and Stewart 2006), velocity pulses (T_P) (Baker 2007), and arias intensity (I_a) (Arias 1970) were also considered when selecting records. The selected records are presented in Table 1.

The records were linearly scaled using the scale factor that caused the mean of the records' FN spectra to exceed the target spectrum within the period range of interest. Three of the records were modified using spectral matching (Al Atik and Abrahamson 2010) to help reduce excessive long-period energy (i.e. when straight scaling resulted in excessive long period spectral acceleration compared to the target spectrum, then spectral matching was used for long periods).

Table 1. Records selected for nonlinear response history analysis.

RSN	T_P (s)*	Event Name	Year	Station	M**	Fault mechanism	R_{jb}*** (km)	V_{S30} (m/s)	Scale Factor
CMS at 1 second									
292	3.3	Irpinia, Italy-OI	1980	Sturno (STN)	6.9	Normal	6.8	382	1.80
764	1.6	Loma Prieta	1989	Gilroy - Historic Bldg.	6.93	Reverse Oblique	10.3	309	3.10
767	2.6	Loma Prieta	1989	Gilroy Array #3	6.93	Reverse Oblique	12.2	350	1.92
828	3	Cape Mendocino	1992	Petrolia	7.01	Reverse	0	422	1.38
3746	2	Cape Mendocino	1992	Centerville Beach, Naval Fac	7.01	Reverse	16.4	459	2.00
4228	1.8	Niigata, Japan	2004	NGHI	6.63	Reverse	6.3	375	2.05
4451	1.4	Montenegr o, Yugoslavia	1979	Bar-Skupstina Opstine	7.1	Reverse	0	462	1.85
4458	2	Montenegr o, Yugoslavia	1979	Ulcinj - Hotel Olimpic	7.1	Reverse	4	319	1.85
4847	1.4	Chuetsu-oki, Japan	1989	Joetsu Kakizakiku Kakizaki	6.8	Reverse	9.4	383	1.44
1258	-	Chi-Chi, Taiwan	1999	HWA005	7.62	Reverse Oblique	43.2	459	4.00
1294	-	Chi-Chi, Taiwan	1999	HWA048	7.62	Reverse Oblique	47.4	346	3.97
CMS at 6 seconds									
723	2.4	Superstitio n Hills-02	1987	Parachute Test Site	6.5	strike slip	1	349	0.70
803	5.6	Loma Prieta	1989	Saratoga - W Valley Coll.	6.9	Reverse Oblique	8.5	348	1.10
900	7.5	Landers	1992	Yermo Fire Station	7.3	strike slip	23.6	354	1.20
1176	4.9	Kocaeli, Turkey	1999	Yarimca	7.5	strike slip	1.4	297	0.80
3744	5.4	Cape Mendocino	1992	Bunker Hill FAA	7	Reverse	8.5	566	1.16
4847	1.4	Chuetsu-oki, Japan	2007	Joetsu Kakizakiku	6.8	Reverse	9.4	383	1.10
6906	6.2	Darfield, New Zealand	2010	GDLC	7	strike slip	1.2	344	0.51
6960	9.4	Darfield, New Zealand	2010	Riccarton High School	7	strike slip	13.6	293	0.90
6962	7.1	Darfield, New Zealand	2010	ROLC	7	strike slip	0	296	0.64
1232	-	Chi-Chi, Taiwan	1999	CHY081	7.6	Reverse Oblique	41.4	573	4.00
1261	-	Chi-Chi, Taiwan	1999	HWA009	7.6	Reverse Oblique	52.4	373	3.46

* Pulse Period, ** Moment Magnitude, *** Boore-Joyner Distance

Evaluation of Impact on Existing Tower

A Perform-3D model was used to evaluate the current condition and the proposed condition, as shown in Figure 9. For the existing condition, the story drift was computed as shown in Figure 10.

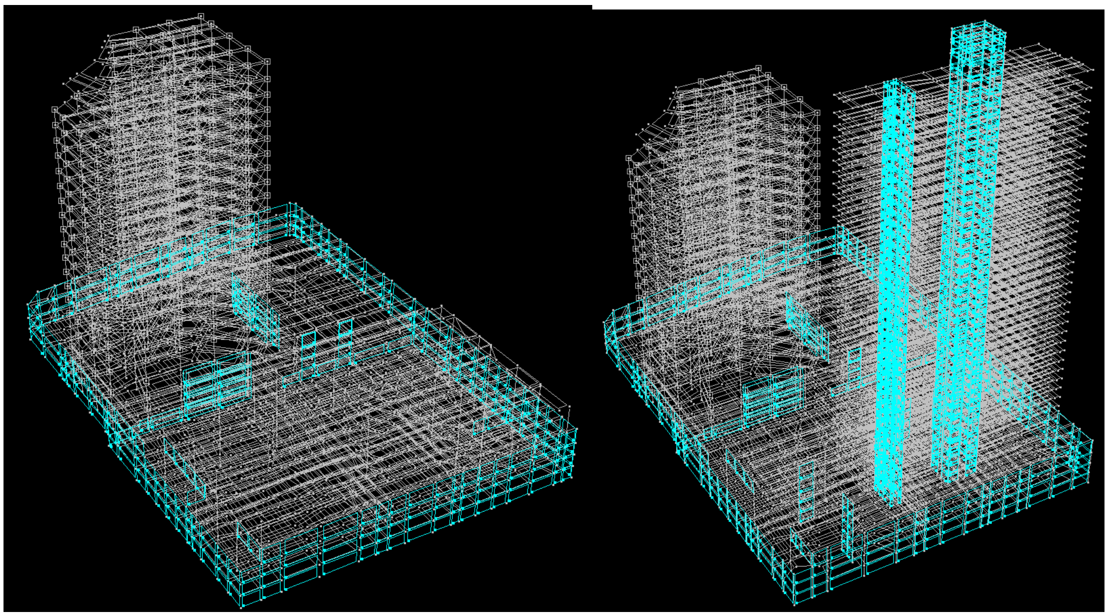

Figure 9 Evaluation of Impact on Existing Structure - Modeled with Perform-3D: a) Prior Condition b) Proposed Condition

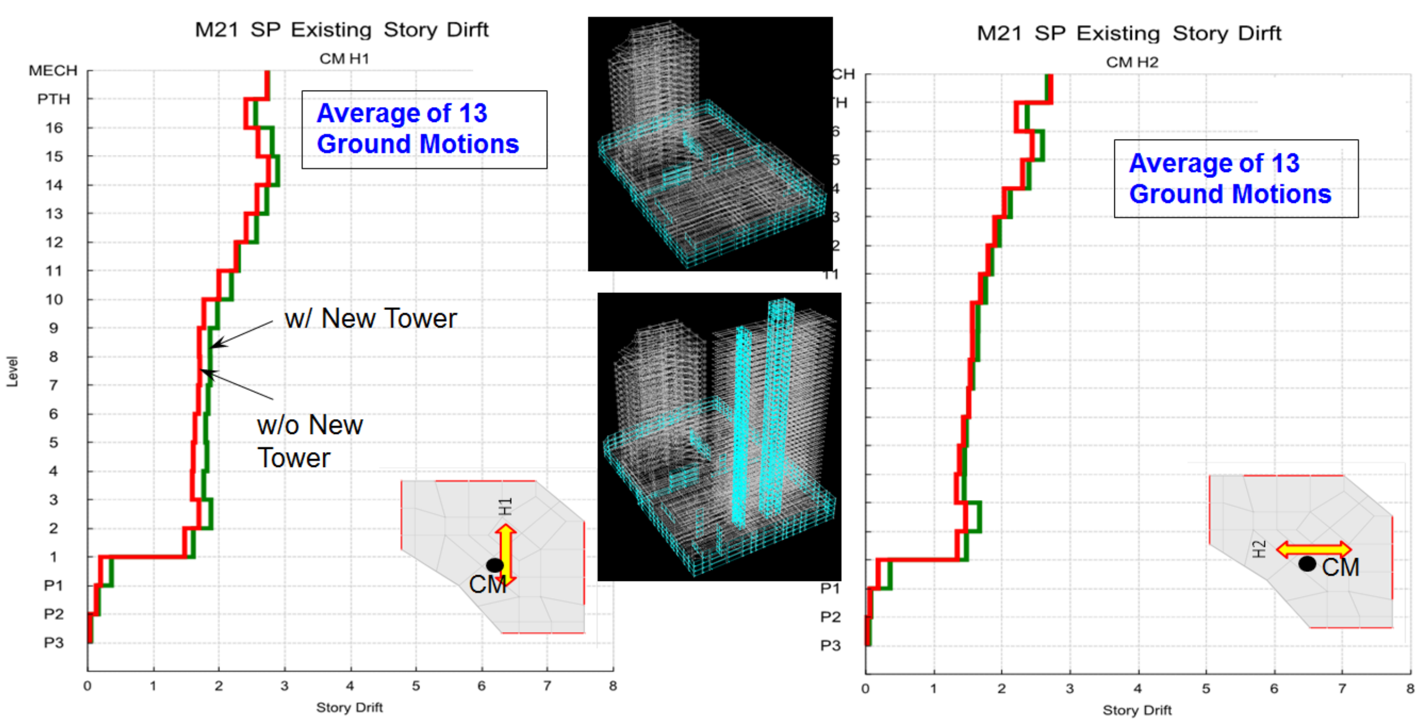

Figure 10 Existing Tower Evaluation - Story Drift

The results of the evaluation indicated that performance of the existing structure would not be significantly changed, and therefore the existing building would not have to be modified as part of the new development, achieving the requirement that the existing tower remain operational during construction of the new tower.

Foundations

The original development was constructed on spread footing foundations. Although the soils have some degree of compressibility, nevertheless considering the pressure relief provided by the depth of excavation for the original basement, the net loads from the 17-story building were significantly less than would be produced by the same building constructed at-grade; therefore, acceptable settlement was predicted allowing use of spread footing foundations. In addition to the spread footings, there were some auger-cast piles as part of the original construction for use only for uplift, where required due to structural loading conditions.

However, the new high-rise had heavier column loads than the existing tower, which would produce greater settlement, and negligible addition excavation was made for the new tower, which meant that the net bearing pressure of footings would be about the same as the gross bearing pressure. In addition, very little settlement would be tolerable with new foundations because new shallow foundation settlements would have also induced some settlement in the existing footings. Therefore, the decision was made to support the new tower on drilled cast-in-drilled-hole (CIDH) pile foundations. The final design of piles resulted in the shafts being 4-ft diameter and 65 to 90 ft long (below pile cap). The pile caps were 6 ft 6 in thick. New pile caps were constructed within the existing basement, with some of the pile caps immediately adjacent to existing spread footings.

In addition to the piles, some existing footings within the existing structure were widened in order to provide additional seismic resistance for the permanent structure, and some new footings were also constructed for shear walls within the existing structure to provide additional seismic resistance (these new or enlarged footings were for those walls/columns not within the footprint of the new high-rise).

Sequence Of Construction

First, the project required demolition of portions of the existing structure, which resulted in reduction in column loads on some of the existing footings. Next, since the pre-existing groundwater level was at the level of the gravel subdrain layer at the base of the slab-on-grade, dewatering was performed to lower the groundwater to the bottom of the deeper planned excavations for pile caps and new footings. Then, excavations were made adjacent to existing footings to allow underpinning of those existing footings to ensure stability during excavation for the planned adjacent pile caps.

Shoring Of Existing Basement Wall

In order to construct the new tower, a “glory-hole” had to be cut through multiple slabs in the subterranean structure, at the location shown in Figure 11. This glory-hole was relatively close to the existing basement wall, which prior to the new construction had lateral restraint provided through diaphragm action of the floor slabs. With the planned removal of a portion of the slabs, the basement wall had to have replacement temporary lateral restraint. The most effective way of accomplishing this would have been to use temporary tie-back earth anchors drilled through the basement wall, tensioned, and anchored to the basement wall. However, for much of the length of the wall, the anchor length was restricted due to property and public right-of-way restrictions. Therefore, a scheme was designed by Plan B Engineering to utilize raker bracing, with the raker bracing supported by micropiles and tie-back anchors, as shown in Figure 12. Figure 13shows a photograph of the temporary bracing system being installed within the existing basement. The need for this temporary bracing system required that the pile cap for the new high-rise be split into two major sections, as shown in Figure 11, in order to accommodate space for the raker brace micropile foundation, with its own tie-back anchors. Tie-back anchors were required at the raker brace foundation because the micropiles have negligible lateral capacity, and the excavation for the permanent pile cap adjacent to the raker micropile cap resulted in no passive pressure being available.

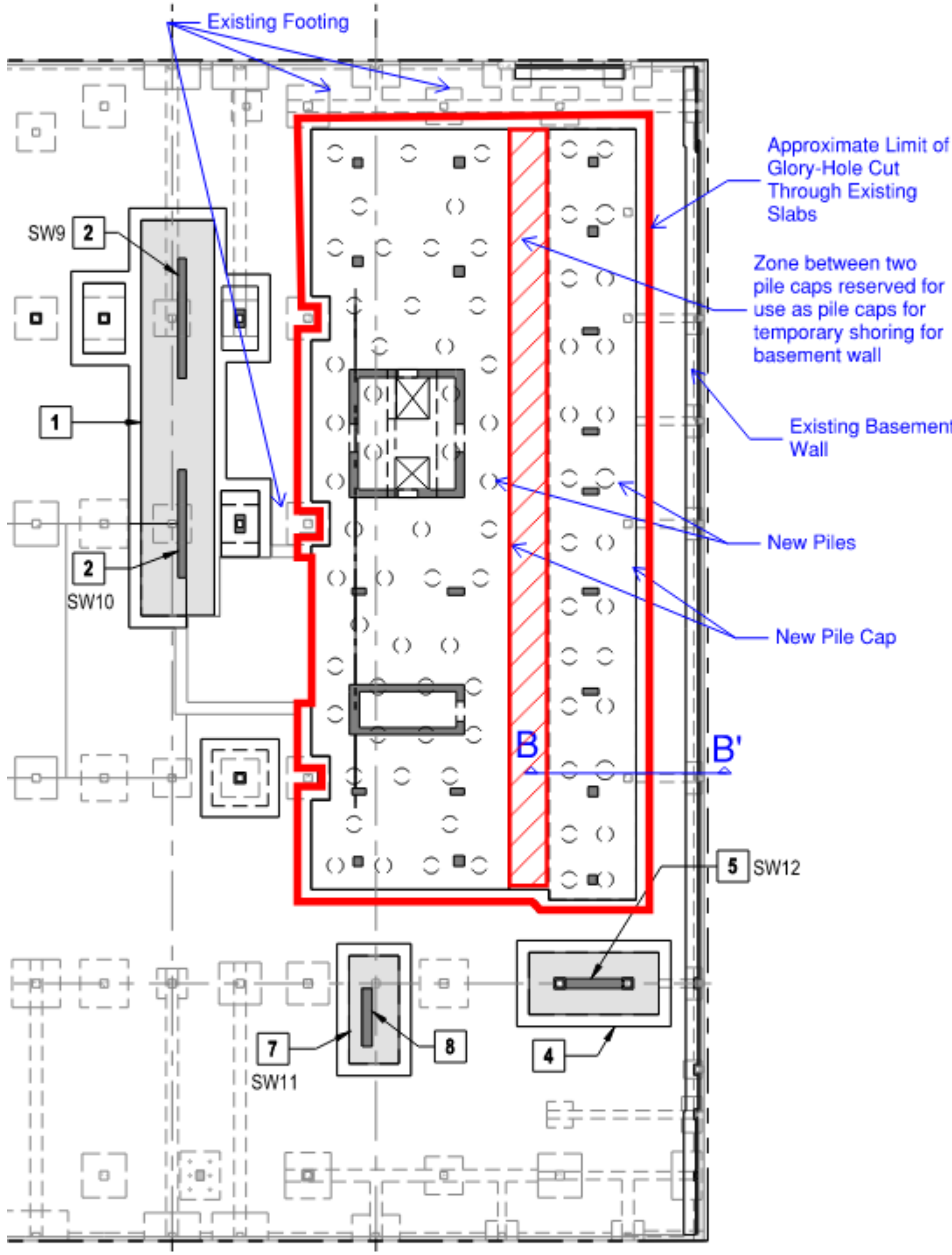

Figure 11 Plan of Existing and New Foundations and Existing Basement Wall

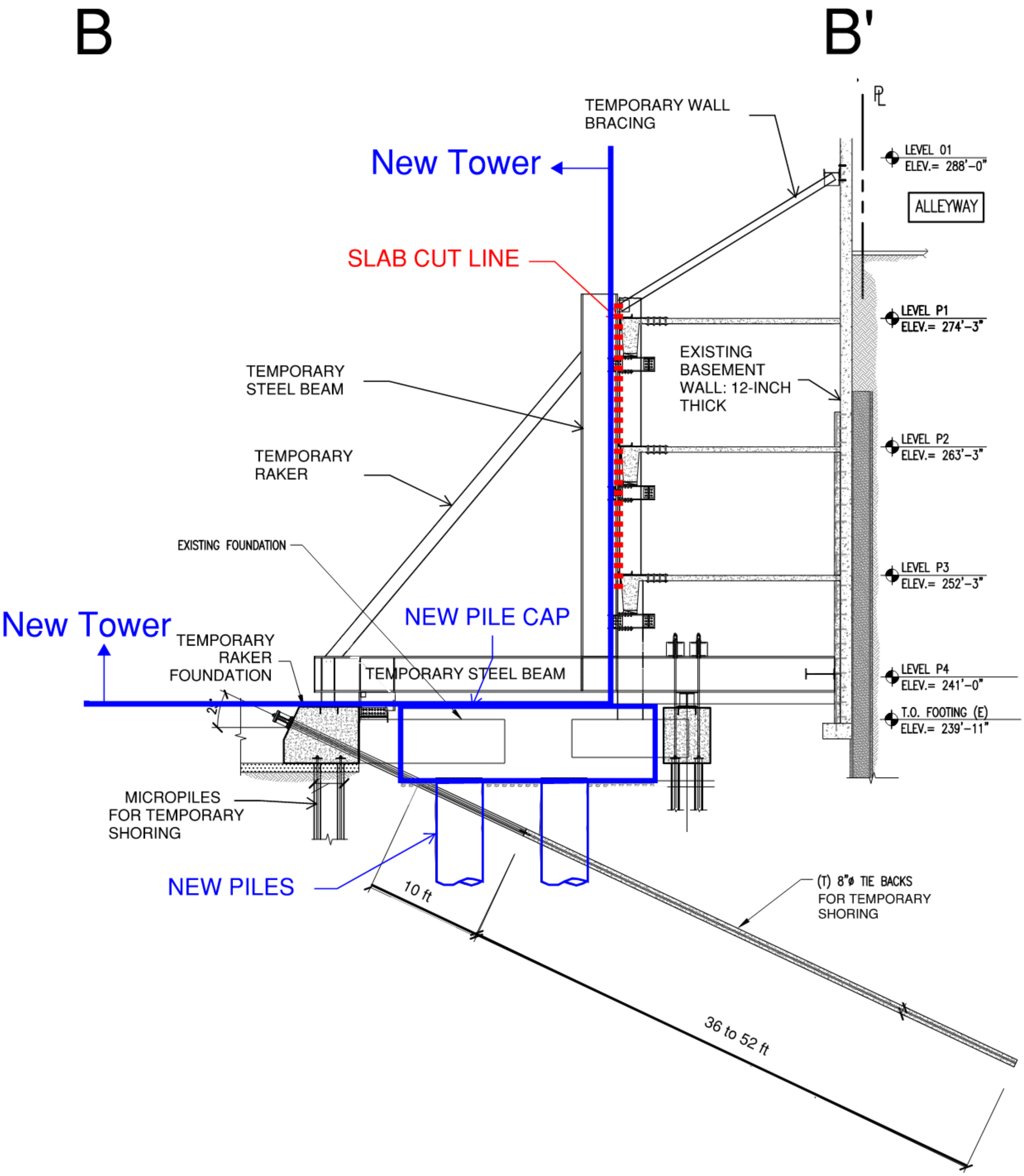

Figure 12 Cross-Section Illustrating Temporary Bracing of South Basement Wall

Figure 13 Temporary Basement Wall Internal Bracing Within Existing Structure a) View Within Basement b) View from Above

Monitoring Of Existing Structure During Construction

During construction, a monitoring program was performed to provide sufficient stability of the existing structures and reduce potential damage to the portions of the existing structure to remain after construction. The monitoring included survey monitoring of the existing structure.

In particular, monitoring was crucial for evaluating the timing for cutting of the existing slabs in the basement which served as diaphragms for supporting the basement walls. First, the shoring and tie-back system was installed within the structure in such a manner that as little of the existing structure as possible was demolished, which required some limited demolition of portions of floor slabs and walls. After the shoring and tie-back system was installed, the tie-back anchors and rakers were pre-loaded to the degree that the load was removed from the slabs. For anchors, the sequence included first loading each tie-back anchor to a test load value, then backing off the anchor load until the deflection of the wall indicated that the full load of the wall was transferred off of the slabs. Then the remainder of the slabs and the interior of the structure could be demolished.

Construction Of New Structural Elements

After construction of new piles and pile caps, the new structure was constructed within the basement, including new diaphragm floor slabs. The new floor slabs, when completed, then allowed the shoring to be unloaded and dismantled, then holes within slabs and walls could be patched where temporary shoring elements had been present.

Conclusions

The construction of a new high-rise within an existing structure required consideration of new foundation settlement, existing foundation settlement, micropile foundations, and a complex basement wall shoring system. The use of multiple companies and engineering disciplines allowed the existing structure to be built within the existing constraints.

Acknowledgements

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