OFFICE OF THE
CITY ADMINISTRATOR

Edwin M. Lee, Mayor
Naomi M. Kelly, City Administrator

PUBLIC NOTICE

301 Mission Seismic Safety Study Committee

Thursday, August 24, 2017

2:30 PM

City Hall

One Carlton B. Goodlett Place, Room 370

San Francisco, CA 94102

The 301 Mission Seismic Safety Committee will hold a meeting to receive the report titled "Structural Safety Review of the Millennium Tower" conducted at the request of Mayor Edwin M. Lee and executed through the office of City Administrator Naomi M. Kelly. The analyses were completed by technical experts Gregory Deierlein, Ph.D., Marko Schotanus, SE, Ph.D., and Craig Shields, PE, GE.

AGENDA

1. Welcome and introductions: City Administrator Naomi Kelly
2. Presentation of Report by technical experts
3. Questions, comments from Seismic Study Committee

A copy of the report is posted with this notice.

For additional information please contact City.Administrator@sfgov.org or call 415-554-4181

July 28, 2017

Naomi M. Kelly
City Administrator
City & County of San Francisco
1 Dr. Carlton B. Goodlett Place, Room 362
San Francisco, CA 94012

RE: Structural Safety of the 301 Mission Street (Millennium Tower) Building

Dear Ms. Kelly:

At the request of the 301 Mission Street Seismic Safety Study Committee, Marko Schotanus, Craig Shields and I have reviewed materials and reports related to the structural safety of the 58-story Millennium Tower building. Attached is our report to the committee.

Sincerely,

Gregory G. Deierlein

Structural Safety Review of the Millennium Tower

Report by:
Gregory Deierlein, Ph.D.
Marko Schotanus, SE, Ph.D.
Craig Shields, PE, GE

Submitted to:
301 Mission Street Seismic Safety Study Committee
City & County of San Francisco

July 28, 2017

EXECUTIVE SUMMARY

The following Structural Safety Review of the Millennium Tower at 301 Mission Street is a summary report by technical experts Gregory Deierlein, PhD, Marko Schotanus, SE, PhD, and Craig Shields, PE, GE, appointed by the City's 301 Mission Street Seismic Safety Study Committee to determine the building's ability to meet structural and seismic safety standards of the San Francisco Building Code.

As of July 2017, the Millennium Tower has settled approximately 17 inches vertically since its construction began in 2006. This settlement has not occurred uniformly, causing a distortion and tilting of the mat foundation, along with a lean (out-of-plumbness) of the building.

This report considers the effects of the tower's significant settlement on the overall safety of its structural and foundation systems against strong earthquake shaking. Its conclusions are based on a review of building design documents, survey information, and results from advanced computer modelling conducted by engineering firm Simpson Gumpertz and Heger (SGH) to simulate the building's response to severe earthquake ground motions.

The Millennium Tower was designed and constructed under 2001 building codes. Following review of an analysis study completed by SGH in 2016, the City-appointed review committee made a number of recommendations to SGH to expand their study, including analysis under earthquake ground motions and criteria that are consistent with the 2016 San Francisco Building Code for similarly designed new tall buildings. SGH issued an updated report in July 2017 concluding that the current foundation settlement of the Millennium Tower has not appreciatively affected the building's safety and that most building elements meet current (2016) criteria adopted by the City and County of San Francisco for the seismic design of similar new buildings.

Based on careful review of the SGH analyses and other materials, the City-appointed technical experts concur with SGH's conclusions that the settlements experienced by the 301 Mission tower have not compromised the building's ability to resist strong earthquakes and have not had a significant effect on the building's safety.

As the building continues to settle at a fairly consistent rate of about 1 inch per year, the report recommends continued monitoring of the Millennium Tower and a reevaluation of the structure when the amount of expected long term settlement has been confirmed.

1.0 INTRODUCTION AND SCOPE

As requested by the City and County of San Francisco's City's 301 Mission Street Seismic Safety Study Committee, the authors have conducted a review of materials to investigate the structural safety of the Millennium Tower building in San Francisco. This report summarizes the findings of the review of the documents, reports and other materials described herein to help the City and County of San Francisco ascertain that the Millennium Tower meets the minimum structural and seismic safety requirements expected for buildings that are designed in accordance with the building code requirements of the City and County of San Francisco and the State of California.

The Millennium Tower is located on the south side of Mission Street, between Beale Street and Fremont Street. The complex consists of two structures, identified as the "Tower" and the "Mid-rise" on the structural drawings (DeSimone, 2006). The structural drawings describe the Tower as a 58-story, 605-ft tall structure over a one story basement, and the Mid-rise as a 12-story, 128-ft tall structure over five below-grade levels. The Mid-rise structure includes the three-story-tall portion between the 12- and 58-story tall towers (note that the first three stories of both structures are sometimes collectively identified as the "Podium" for functional, rather than structural reasons). The Tower and Mid-rise are structurally separated by a seismic joint.

Both structures are of cast-in-place concrete construction, using post-tensioned slabs for the floors above ground level.

The seismic force-resisting system of the tower consists of a "Dual System", which is comprised of a 36-inch-thick special reinforced concrete shear wall core with outriggers and concrete special moment-resisting frames. The mid-rise relies on a special reinforced concrete shear wall system that includes the perimeter basement walls.

The two structures use different foundation systems. The Tower foundation consists of a 10-foot-thick pile cap supported by about 950 precast concrete piles, measuring approximately 80 feet in length. The Mid-rise structure rests on a mat foundation that varies between 6 and 8 feet in thickness. Tie-downs resist hydrostatic uplift pressures under the three story portion of the Mid-rise building. The original design anticipated 1 inch of settlement under the Tower by the time of construction completion, and additional long-term settlement due to compression of the underlying clay layers of 5 inches. Settlements were expected to occur uniformly over the Tower foundation area.

Reports of the large vertical settlement, differential settlement and tilting of the Tower structure have raised concerns regarding the structural integrity of the building and its safety against earthquakes. The focus of this review is on the structural system of the Tower.

Note that while this review has considered the implications of the adjacent Mid-rise building and associated mechanical, electrical and architectural components of the Tower, the review is primarily concerned with the structure and foundation of the Tower itself. Further, the review is limited to evaluation of the current condition of the Tower and does not address the effects of future settlement or other changes that may occur to the Tower in the future. Nor does the report attempt to address the causes of the settlement or implications of the settlement on the serviceability of the building.

2.0 PROCESS AND MATERIALS REVIEWED

This review has relied upon building design drawings, building survey information, and analysis reports that have been provided by the City and County of San Francisco. The subset of materials that we reviewed, which are most directly related to this report, are listed below (with full citations in the appendix to this report):

1. Structural design drawings of 301 Mission Street (DeSimone, 2006).
2. Geotechnical investigation for 301 Mission Street (Treadwell & Rollo, 2005)
3. Summary of Pile Driving (Treadwell & Rollo, 2006)
4. DeSimone Consulting Engineers correspondence with the San Francisco Department of Building Inspection regarding settlement (DeSimone, 2009)
5. ARUP surveys of settlement (ARUP, 2009, 2016 and 2017)
6. Inspection Division Staff Report – 301 Mission Street, San Francisco Department of Building Inspection (SFDBI, 2017)
7. SAGE Engineers investigation of settlement of 301 Mission Street Tower (Sage, 2016)
8. Building engineering report and safety evaluation by Millennium Tower Association (Duane Morris, 2016)
9. Survey of building out-of-plumb (Langan, 2017a, 2017b)
10. SGH investigation reports of the seismic performance of the 301 Mission Street Tower (SGH 2014, 2016, 2017a-f)
11. Allana Buick & Bers report of Site Utilities Investigation (Allana, 2017)
12. Journal paper on building seismic instrumentation (Celebi, 2016)

While we have carefully reviewed this information, we have not independently verified surveys or analysis results.

A primary source of information that we relied upon to assess the earthquake safety of the building is the study performed by Simpson Gumpertz and Heger (SGH), under supervision of Ronald Hamburger, SE, who have been engaged by Millennium Partners to investigate the earthquake safety of the Tower. Based on an initial review of their October 2014 and October

2016 Foundation Settlement Investigation reports (SGH 2014, 2016), we made a series of comments to SGH, which identified some questions and gaps in their analysis (such as incomplete assessment of the earthquake loading demands on the precast piles). Two meetings were convened to discuss modeling assumptions, acceptance limits, and results. The recent SGH update to the two earlier reports (SGH, 2017) incorporates analyses and conclusions that resulted from these discussions.

In addition, we met once with an engineering team led by Leslie E. Robertson and Associates (LERA), who have been hired by the Millennium homeowners association to develop possible retrofit methods to mitigate possible future settlement of the Millennium Tower.

Finally, two of us (Deierlein and Schotanus) visited the Millennium Tower on December 14, 2016, during which time we inspected the basement of the Tower and adjacent Mid-rise, the concrete core walls in the Tower, the rooftop machine room of the Tower, and the perimeter of the site.

3.0 OVERVIEW OF TOWER DESIGN

According to the Building Permit History, Building Inspection History, and the submitted design drawings, the Millennium Tower was designed in accordance with the 2001 San Francisco Building Code and described in design documents and construction permits issued between 2005 and 2009. The building construction commenced in 2006 and was completed in 2009.

A general review of the structural design drawings and the subsequent structural analysis studies described later indicate that the building generally conforms to the building code provisions and that the structure employs some features, which go beyond the minimum building code requirements in effect at the time. In particular, the structural design drawings indicate that the wall outrigger system is designed using capacity design principles, whereby the strengths of the outrigger columns and supporting mat are larger than the strengths of the outrigger beams. The outrigger beams are also designed with confining reinforcement to improve their post-yield behavior, which can occur under large earthquakes. In addition, the concrete mat foundation includes vertical shear reinforcement, which helps in the redistribution of forces from the concrete shear walls and columns into the supporting piles.

The building has seismic instrumentation consisting of 32 accelerometers on 10 levels, installed in 2009 as part of the California Strong Motion Instrumentation program. The instruments recorded acceleration during the Mw 3.8 Berkeley earthquake of 20 October 2011 and the Mw 6.0 Napa earthquake of 24 August 2014. Analysis of the recordings (Celebi, 2016) indicated primary vibration periods of 3.8s in the NS direction and 4.0s in the EW direction. These vibration periods are within the range expected for a building of this height and structural system type. Analysis of data measured during the Napa earthquake indicates that the drifts experienced during the earthquake were small (roof displacements less than 1.2 inches or 0.015% during the Napa earthquake) and that rigid body motions due to foundation rocking are not significant. While these data are not necessarily indicative of what may happen under larger earthquakes, they provide some assurance as to the integrity of the building and foundation.

4.0 BUILDING CODE INTENT AND PROVISIONS FOR SEISMIC SAFETY

Modern building code requirements for structural design are primarily intended to minimize the risk of structural damage and collapse that would endanger building occupants under earthquakes or other extreme loadings (e.g., high wind loads, excess floor live loads, etc.). While the minimum building code design requirements may help control damage under small (frequent) earthquakes, the requirements offer no assurances that buildings will be habitable or even repairable following extreme earthquakes. Moreover, as building code requirements are highly prescriptive, they do not provide an explicit measure of risk, except insofar that building code requirements have evolved to provide a level of safety that is generally acceptable to society.

As described later, the safety of the Millennium Tower and the potential impact of foundation settlement on the building safety are evaluated based on detailed computer structural analyses, conducted by SGH, along with a qualitative review of information related to the design along with measurements and observations of the building settlement and other information. The computer analyses employed in the study by SGH are nonlinear dynamic analyses, which are intended to simulate the response of the building due to ground motions from a large earthquake, similar in magnitude to the 1906 San Francisco earthquake.

5.0 SUMMARY OF CURRENT CONDITION

5.1 Building Settlement: Survey reports by ARUP (2009, 2016, 2017) indicate that the building had settled about 6 inches by the time of the Tower structure completion (February 2008). The total settlement increased to about 9 inches by the project's completion (August 2009) and subsequently increased to about 16 inches (June 2016) and almost 17 inches, presently (July 2017). In addition to vertical settlement, the building mat foundation has experienced some distortion (dishing) and tilting. According to the latest ARUP survey (July 2017), the maximum difference in elevation across the tower mat foundation is about 6 inches. Comparative measurements of mat elevations from April 2009 to July 2017 indicate differential settlements of about 1 inch from the south to north end of the mat and about 2 inches from east to west across the mat, during this time. In contrast to the total vertical settlement and rigid body tilt that continue to increase, the surveys indicate that most of the mat distortion occurred by 2009.

The mat tilting is accompanied by a building lean of about 0.18% of the building height, with total horizontal roof displacements of 14.0 inches to the west and 6.3 inches to the north (Langan, 2017b). As a point of reference, the maximum construction tolerance for out-of-plumbness (ACI 117-10) is 1/600 times the building height (about 0.17%) for buildings taller than 100 ft and is limited to 6 inches total.

Observations of the site conditions, geotechnical reports (Treadwell & Rollo, 2005, SAGE, 2016), building foundation drawings and settlement measurements indicate that the primary mechanism for the large vertical settlement is consolidation of the Old Bay Clay that exists at depths of roughly 90 to 220 feet beneath the ground surface. These Old Bay Clay layers underlie the Marine Sands (occurring at depths of 40 to 90 feet) into which the precast piles are driven. The deep-seated settlement occurs primarily below the building but extends gradually outside the footprint of the tower foundation. The consolidation of the clay layers is a relatively slow process, occurring over a period of years, due to the increase in effective stress in the Old Bay Clay layers. This understanding as to the mechanism of the settlement is important to help confirm that the settlement is not due to distress in the foundation piles that may affect their ability to sustain forces associated with gravity and earthquake loading demands.

As indicated in the soil report by Treadwell & Rollo (2005), the layers of saturated, loose to medium dense sand beneath the Tower foundation are susceptible to liquefaction during a moderate to large earthquake. They estimate that liquefaction-induced settlement of these layers may be on the order of 1 inch, although estimates of this settlement can be highly variable. As noted by Treadwell & Rollo, being as the precast piles extend through the liquefiable layers, the liquefaction-induced settlement of the upper soil layers are not expected to affect the tower or its foundation. However, as also noted in their report, liquefaction may cause significant subsidence of streets and sidewalks around the building, which can have implications on the building's buried utility lines.

5.2 Visible Damage: As noted in the ARUP report (2012) and seen during our site visit, cracking has been observed in the perimeter basement walls of the tower. ARUP installed 119 gauges to monitor crack growth, including 103 installed in April 2009, 15 in May 2011 and 1 in December 2011. As indicated in their 2012 report, as of January 2012, most of the gauges had recorded less than 0.1 mm of crack movement between 2011 to 2012. Twenty-five had experienced larger movement, on the order of 0.5 to 1.0 mm of additional crack opening.

In the Tower, the lateral system consisting of the concrete core shear wall and moment frame columns are fully supported by the 10-foot-thick foundation mat, which is supported by the precast piles below. As such, the single-story perimeter basement walls are not part of the lateral force resisting system for wind or earthquake. Rather, the main purpose of the perimeter walls is to resist the local earth pressures and to provide vertical support to the grade-level slab around the building perimeter. Thus, while the cracks in the perimeter walls may affect groundwater infiltration, which if left unremedied could lead to corrosion and local failure of the walls, the cracks are not a significant concern with the overall structural integrity of the tower.

During our site visit, we rode atop one of the elevators to inspect the exposed interior of the concrete shear walls in the building core. While we did not inspect the entire core, the region that we did inspect did not exhibit excessive cracking or other damage. Only a few local hairline cracks were observed consistent what is expected from concrete shrinkage during curing. We also observed a few localized patches of honeycombing (small voids in the concrete) that were most likely present since the concrete was cast and are not of any significance in the dry elevator shaft environment.

During the site visit, we also observed more extensive concrete cracking and spalling in the basement walls of the neighboring 12-story Mid-rise building. As the Mid-rise building is structurally separated from the Tower building by a seismic gap, the cracking in the basement walls of the Mid-rise has no effect on the Tower. At the time of our visit, work was underway to fix the basement walls of the Mid-rise by chipping out and replacing the damaged concrete.

6.0 REVIEW AND OBSERVATIONS OF SGH ANALYSIS STUDY

Apart from the general impressions regarding the building design and the settlements that it has experienced, our conclusions regarding the earthquake safety of the building are largely based on a detailed review of the analyses performed by Simpson Gumpertz and Heger (SGH), under supervision of Ronald Hamburger, SE (SGH, 2017a-f). The final report (SGH, 2017f) represents a major update to two earlier reports (2014 and 2016) that includes additional analyses and conclusions in response to concerns that we raised. Based on their analyses, the SGH team concludes that: (1) the effect of settlement on most building elements is negligible, (2) under the influence of Maximum Considered Earthquake shaking together with the settlements that have occurred to date, most building elements continue to meet criteria commonly adopted for design of similar new buildings in the city of San Francisco, and (3) the settlements experienced by the 301 Mission tower have not compromised the building's ability to resist strong earthquakes and have not had a significant effect on the building's safety. As outlined below, based on our review of the SGH analysis study, we accept their conclusions.

SGH's engineering team investigated the building's performance by analyzing the structural system and foundations using nonlinear response-history analysis (NLRHA) under severe earthquake ground motions. This approach to establishing acceptability of seismic safety is consistent with current requirements and standard practice for tall buildings in San Francisco that are designed using non-prescriptive seismic design procedures, as described in the San Francisco Department of Building Inspection's (SFDBI) Administrative Bulletin AB-083 (2011) and associated guidelines and standards (PEER, 2010, ASCE 7-10).

The SGH analysis model incorporates all major structural components of the building, including the concrete core wall, outrigger beams and columns, concrete moment frames, floor diaphragms, the mat foundation, and supporting piles. In addition to the gravity loads due to the self-weight and contents of the building, the analysis model includes imposed deformations and induced forces caused by the building settlement. The SGH model conservatively applies all of the current measured settlement on the complete building model. The model is analyzed under seven pairs of bi-directional earthquake ground motions that represent the so-called Maximum Considered Earthquake (MCE) at the 301 Mission Street site. This ground motion intensity has a peak horizontal ground acceleration of 0.6g and a spectral acceleration of about 0.2g at the fundamental mode of vibration of the building.

Based on our initial review of the October 2014 and October 2016 Foundation Settlement Investigation reports by SGH, we submitted a series of comments and questions to SGH, and we met with them twice to discuss modeling assumptions, acceptance limits, and results. The main focus of our discussions was on: (1) the level of ground shaking applied in the nonlinear analyses, (2) the modeling of settlement and out-of-plumb, (3) the modeling and checking forces in the pile foundations, (4) the modeling of the outrigger beams, and (5) the impact of undesirable structural component behavior identified by the advanced analysis. In addition, whereas the prior studies focused on the changes in response caused by the settlement, in the latest set of analyses we requested that there be more attention to assessing the overall response relative to criteria that would be acceptable for tall buildings designed according to AB-083 and associated design guidelines.

Our review of the SGH model indicates that it conforms to state-of-the-art engineering practice and is properly calibrated to represent the properties of the building. The model was created using the nonlinear analysis software system CSI-Perform-3D V5.0.1 (CSI, 2017), which has been used in the performance-based design of most of the tall buildings that have been built in San Francisco and other west-coast cities over the past decade. The induced pile support settlements in the computer model match closely the survey measurements of June 2016 (ARUP, 2016), and the resulting building out-of-plumb is generally consistent with building survey measurements (Langan, 2017a). The fundamental vibration periods in the two orthogonal directions of the computer model are about 5s, which are consistent with expected values for a 58 story building. Note that these periods are larger than the vibration periods of 4.0s and 3.8s measured in the building during the Napa earthquake, since the computer model conservatively neglects the contribution of secondary structural effects and architectural walls and cladding and is calibrated to represent building response under stronger ground shaking. Modeling parameters for structural components of the buildings were developed based on accepted standards for nonlinear analysis, such as ASCE 41-13.

Based on our discussions, SGH agreed to conduct their latest set of analyses with ground motions (earthquake demands) that are consistent with the Maximum Considered Earthquake (MCE) hazard, as specified in the current 2016 San Francisco Building Code, based on ASCE 7-10. The MCE hazard represents severe earthquake effects that are specified by the United States Geologic Service (USGS). For the 301 Mission Street site, the MCE ground shaking intensity is defined based on an estimate of shaking for an earthquake of Moment Magnitude 8 on a nearby segment of the San Andreas fault (roughly equivalent to the 1906 earthquake). The forces and deformations imposed on the building by this level of shaking are larger than the demands required for the original design of the structure, based on the 2001 San Francisco Building Code, but they are consistent with the safety evaluation based on current building code provisions for tall buildings.

While the nonlinear analyses reported in the 2016 SGH report included modeling of the mat foundation and piles, the model did not allow for calculation of forces induced in the individual piles. Based on our discussions, SGH agreed to perform a more detailed analysis that would allow for explicit evaluation of the resulting forces (axial force, shear and moment) in the piles. The resulting analyses, documented in their 2017 report, confirmed that the vertical and horizontal pile demands do not exceed the force and deformation limit states in the piles, accounting for redistribution of loads resulting from the settlements. Development and evaluation of the pile analyses included additional review of pile driving records to establish appropriate pile strength and stiffness that accounts for as-built pile lengths and local site soil conditions.

Because the Millennium Tower was not designed based on advanced performance-based methods that employ NLRHA, the NLRHA identified some less desirable behavior as would be commonly expected. For this structure, this includes shear yielding of the core wall and flexural yielding of moment frame columns. In general the inelastic strains in these components are well distributed with peak values below acceptance limits, and therefore, they are not considered to negatively affect building safety. The analyses also identify that the inelastic displacement demands on the outrigger coupling beams exceed acceptance criteria. Based on our discussions, SGH modified the outrigger beam models to represent strength and stiffness degradation under large inelastic cyclic loading, enabling the analysis to reliably capture their reduced capacity during the analyses. In this way, the analyses captured the peak deformations in the structure considering this degradation.

The following is a summary of the key results of the SGH investigation that were used to establish that the Tower exhibits seismic performance that is relatively unaffected by the foundation settlement and is comparable with the building code intent for tall buildings:

Peak Story Drifts: Peak story drifts are one of the primary criteria applied in AB-083 and other performance-based provisions for tall buildings. AB-083 limits the average peak story drift ratios to 3%. The SGH study confirmed that under the MCE level ground motions, the average peak drift under the seven ground motion pairs is 2.2%, which is considerably less than the limit of 3%. Their study further confirms that (1) the maximum of the seven ground motion pairs has a peak drift of 3%, which is less than the peak value of 4.5% permitted in the PEER guidelines for tall buildings (2010), and (2) the average residual drift is about 0.3%, which is less than the suggested limit of 1% in the PEER guidelines (2010).

Flexural and Shear Strains in the Concrete Core Walls: Compressive and tensile flexural strain demands in the concrete core walls are considerably less than the acceptance criteria (average peak compressive strain demands are less than about one half of limit of 0.011 and tensile strains are less than about one-tenth of the limit of 0.050). The average inelastic shear strains in the wall are less than about one-third of the limit of 0.01.

Deformations in the Outrigger Beams: SGH had previously noted the large deformations in the outrigger coupling beams that exceeded acceptance criteria (SGH 2014, 2016). This is not unexpected, since the outrigger coupling beams in the building are based on prescriptive building code design provisions, which were commonly employed in tall buildings prior to about 2010. The prescriptive design approach can result in outrigger beams that are stiffer and, therefore attract larger forces and deformations, than outrigger beams in tall buildings designed after 2010 that employ performance-based design methods. As noted previously, SGH modeled the outrigger beam component to conservatively simulate the potential degradation that occurs at deformations larger than the standard acceptance criteria of the coupling beams.

In the final 2017 report, the calculated deformation demands in the outrigger beams still significantly exceed the shear strain acceptance criteria, which are set at 0.025. In spite of the localized large deformations and degradation in the outrigger beams, the SGH analyses confirmed that the overall building drifts and other measures are within the acceptance criteria.

Deformations in Embedded Steel Coupling Beams: Inelastic rotations in the embedded steel coupling beams, which provide coupling action along the NS shear walls, are within the rotation acceptance criteria of 0.03 radians.

Deformations in Moment Frame Beams and Columns: Inelastic rotations in the special moment frame beams are all significantly (almost an order of magnitude) below their plastic hinge rotation limits of 0.03 to 0.05 radians. This is not surprising given the moderate earthquake drift demands. Plastic hinge rotations of the moment frame columns are somewhat larger, especially at the base of the columns where they are flexurally fixed to the mat. But, even in these locations, the imposed deformations are within the rotation limits of 0.008 to 0.009 radians.

Deformations in Mat Foundation: As noted previously, in the nonlinear analysis, all of the settlement deformations in the mat are imposed on the complete structural model (representative of the fully constructed building). To the extent that the settlements occurred during construction, the analysis model may overestimate the stresses and strains that the settlements cause in the structure. This was done intentionally to provide a conservative estimate of the settlement effects. Even with this conservative approach, the calculated average flexural deformations in the mat are all less than half of the acceptance criteria of 1.0%. As expected, some flexural yielding and inelastic rotation in the mat occurs under the effects of the settlement distortion. The mat deformations increase only modest amounts under earthquake effects, which is attributed in part to the capacity design approach used in the original design of the mat (DeSimone, 2006).

Axial Strains in the Outrigger Columns: Axial strain demands in the outrigger columns are checked to confirm that the columns can sustain the forces imposed by the outrigger beams. The SGH report confirms that the maximum imposed axial strains are less than about 0.002, which is well within the acceptance criteria (strain less than 0.011). This data helps confirm that the structural response is consistent with the capacity design approach employed by the designers for the outrigger columns (DeSimone, 2006).

Axial Forces in the Piles: As expected, the nonlinear analyses indicate that the differential pile settlement causes a significant redistribution of axial forces under gravity loads. This redistribution is enabled by the large strength and stiffness of the concrete mat, combined with the fact that the compressive strength of the piles is limited by the soil resistance (as opposed to the crushing strength of the pile itself), such that any of the piles that are heavily loaded in compression will naturally redistribute loads to neighboring piles. Results of the analyses under combined settlement and gravity loads indicate that the piles can sufficiently resist the applied loads, which is further substantiated by the pattern of the observed building settlements. Results of the NLRHA under the combined effects of settlement, gravity and earthquake ground motions further indicate that the piles have sufficient compressive and tensile strength to resist the imposed forces that would occur during the MCE ground shaking. The analyses of pile capacities under earthquakes takes into account the fact the effective stiffness of the soil is higher during the transient earthquake loading than during long-term gravity loading.

Shear and Bending Forces in the Piles: In addition to checking the axial capacity of the piles, SGH conducted detailed analyses to confirm that the piles can resist the lateral forces that develop during earthquakes. The analysis conservatively assume that all of the inertial earthquake force is resisted by shear and bending in the piles, ignoring the possible resistance by soil shear friction and bearing against the mat foundation. The pile analyses consider lateral resistance of the soil and variability in the pile fixity into the mat foundation. The results confirm that the calculated lateral resistance of the pile foundation (25,000 kips) is larger than the average peak base shear (21,000 kips) calculated in the NLRHA of the overall building.

Finally, as noted above, the SGH study is based on a survey of mat settlements conducted in June 2016 (ARUP, 2016). A more recent survey (Arup, 2017) indicates the mat foundation has settled about 1 inch more over the past year and the differential settlement has caused an additional tilt of the mat of about 0.04% towards the west. SGH has addressed the potential effects of this additional settlement on the structural behavior, considering the results of their previous analyses conducted with varying amounts of settlement, concluding that the additional settlement would not cause any significant impact on their results (SGH 2017f). We concur with their conclusion.

7.0 CONCLUSIONS

The 58-story high rise portion of the Millennium Tower complex has settled about 17 inches vertically since construction commenced in 2006, with about 11 of the 17 inches occurring since completion of the tower superstructure in 2008 (ARUP, 2017). This is more than three times what was anticipated over the life of the structure in design (Treadwell & Rollo, 2005). Though the rate of settlement is less than the peak rates observed during construction, the measurements indicate that the settlement has been progressing at a fairly constant rate of about 1 inch per year since 2010 (ARUP, 2017).

The settlement has not occurred uniformly, resulting in some distortion (dishing) and tilting of the mat foundation (ARUP, 2009, 2016, 2017), some of which continues to occur. Independent measurements of the building out-of-plumb indicate that the building is tilting in the north-west direction. Current measurements show an out-of-plumbness at the top of facade of about 14 inches to the west and 6 inches to the north. This is twice what would be considered an acceptable construction tolerance for out-of-plumb. The continuing differential mat settlement and building tilt are obviously related.

Because of the unexpected and excessive settlement and building tilt, and the lack of stabilization of the settlements, the building warrants in-depth investigation.

Various detailed studies (ARUP, SAGE) indicate that the settlement is occurring in the Old Bay Clay layer below the bottom of the pile foundations supporting the Tower. Therefore, it does not appear that the settlement has affected the ability of the foundation piles to support the structure under earthquakes. The detailed study by SGH (SGH, 2016, 2017a-f) has further demonstrated that, at this time, the settlements and lean have caused only minor changes in the loads experienced by the Tower's structure and no appreciable change to the Tower's capability of resisting major earthquakes.

The SGH study has further demonstrated that in its current condition, the structure generally meets the performance-based design criteria for the seismic design of new tall buildings using non-prescriptive seismic-design procedures. The nonlinear structural analysis procedures employed in this study have been calibrated to consider the effects of degradation that may occur to important structural components under strong ground shaking. Results of these analyses indicate that the structure meets the earthquake building drift criteria of AB-083 (2011) under the Maximum Considered Earthquake ground motions, as specified in the 2016 San Francisco Building Code.

Concerns about the functionality of the building mechanical and electrical systems have also been considered. In general, provisions that had been made during design to accommodate differential settlements between the Tower and the Mid-rise, and between the building and the street, appear to have resulted in satisfactory performance to date.. Inspections by the SFDBI (SFDBI, 2017) and the Millennium Tower Association (Duane Morris, 2016, and Allana, 2017) identified a few issues that either have been or are in the process of being addressed.

To an extent consistent with the scope of our review, our professional opinion is that the foundation settlement experienced by the Tower has not appreciatively affected the safety of the building at this time. However, because the structure is still settling, continued monitoring and further study of the cause of the settlements is recommended to allow a better understanding of maximum future settlements. Once it has been confirmed that the settlement rate will reduce and estimates of maximum long-term settlement and lean are available, reevaluation of the acceptability of the structural performance of the building is recommended.

REFERENCES

AB-083 (2011), *Requirements and Guidelines for the Seismic Design of New Tall Buildings Using Non-Prescriptive Seismic-Design Procedures, Administrative Bulletin*, Updated 1/1/11 for 2010 San Francisco Building Code.

ACI 117-10 (2010), *Specification for Tolerances for Concrete Construction and Materials and Commentary*, Reported by ACI Committee 117, American Concrete Institute.

Allana (2017), *The Millennium Tower Site Utilities Investigation*, Allana Buick & Bers, Inc., June 30, 2017.

ARUP (2009), *Memorandum: Results of Settlement Surveys at 301 Mission Property*, ARUP, October 15, 2009.

ARUP (2016), *Memorandum: Results of June 2016 Settlement Survey at 301 Mission Property*, ARUP, June 16, 2016.

ARUP (2017), *Memorandum: Results of June 2017 Settlement Survey at 301 Mission Property*, ARUP, June 9, 2017.

ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard ASCE/SEI 7-10.

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