DRAFT

Foundation Settlement Investigation

301 Mission Street
San Francisco, CA
27 October 2014

SGH Project 147041

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

PREPARED FOR:

Paul Hastings, LLP
55 Second Street, 24th Floor
San Francisco, CA 94105

PREPARED BY:

Simpson Gumpertz & Heger Inc.
100 Pine Street, Suite 1600
San Francisco, CA 94111
Tel: 415.495.3700
Fax: 415.495.3550

27 October 2014

Mr. Stephen C. Hood
Millennium Partners
735 Market Street, Suite 302
San Francisco, CA 94103

Project 147041 – Evaluation of Effects of Foundation Settlement, 301 Mission Street, San Francisco, CA

Re: Transmittal of Draft Report

Dear Mr. Hood:

We are pleased to transmit the attached draft copy of our report on the subject project. Please review the report and let us know if you have any questions or comments on it. We would be happy to present its salient findings to you, at your convenience.

Sincerely yours,

Ronald O. Hamburger, SE
Senior Principal
CA License No. 2951 (Structural)
I:\SF\Projects\2014\147041.00-301M\WP\001ROHamburger-T-147041.00.jdi_Transmittal_Draft.docx

Encl.

cc: Ms. Cristina Mor

T A B L E   O F   C O N T E N T S

ABSTRACT

Millennium Partners retained Simpson Gumpertz & Heger Inc. to conduct an evaluation of the effect of site settlement on the building at 301 Mission Street, San Francisco, California. Specifically, we were asked to evaluate whether differential settlement experienced by the building has compromised the structure's earthquake resistance.

The building comprises a fifty-eight-story, reinforced concrete residential tower and an adjacent twelve-story podium structure constructed in the period 2007-2010. The tower is founded on a thick reinforced concrete mat supported by precast concrete piles. The podium is supported on shallow foundations. The site is deeply underlain by saturated clays and silty clays deposited over thousands of years by the San Francisco Bay, which prior to the mid-1800s, extended over the site. Geotechnical investigations performed in support of the building design projected site settlement in the range of 4 to 6 in. due to consolidation of the underlying clay materials under the influence of the effect of site dewatering and construction activity. The duration of site dewatering was longer than originally forecast and settlement during construction exceeded originally predicted levels. Following construction completion, construction of adjacent projects resulted in additional site settlements. As of 2014, total settlements exceed 14 in. on some portions of the site, with differential settlement across the tower footprint of more than 6 in.

We reviewed available documentation of the original design and the settlements that have occurred over time; conducted site visits to observe the condition of exposed structural elements; and, performed linear and nonlinear structural analyses of the tower structure subjected to dead, live, settlement, and earthquake-associated demands.

We determined that the effect of settlement on most building elements is negligible. Under the influence of Maximum Considered Earthquake Shaking together with the settlements that have occurred, most building elements continue to meet criteria commonly adopted for design of similar new buildings in the City of San Francisco today. A few elements have somewhat larger demands than commonly considered appropriate for the new design of new tall buildings, even without consideration of the effects of settlement. Settlement generally increases these demands by less than 10% and does not have significant impact on the building's safety.

The building's foundation mat was designed with the intent that it would experience damage, in the form of plastic hinging, when the building is subjected to Maximum Considered Earthquake shaking. The settlement increases the amount of plastic hinging occurring in the mat significantly. In our opinion this does not represent a significant life safety risk.

Under the building code, it is anticipated that when buildings experience strong earthquake shaking they will experience significant damage but have low risk of collapse. We believe the effects of settlement have not compromised the 301 Mission Street building's ability to perform in this manner. However, it is somewhat more susceptible to damage from earthquakes than it would have been had the settlement not occurred, particularly in the foundation mat. It may be feasible to reinforce the mat, to reduce the potential for such damage, by installing new reinforced concrete walls in the basement, extending outward from the central core.

SUMMARY OF FOUNDATION SETTLEMENT INVESTIGATION
301 MISSION STREET, SAN FRANCISCO, CALIFORNIA

1. INTRODUCTION

1.1 Background

The building at 301 Mission Street, San Francisco, California, also known as the Millennium Tower, is a fifty-eight-story, reinforced concrete structure developed by Millennium Partners in 2007 for sale as residential condominium units. The building is located at the southeast corner of Mission Street and Fremont Street. The building comprises two separate structures, the fifty-eight-story tower and an adjacent, functionally connected, twelve-story reinforced concrete podium.

Prior to the era of the California gold rush, the 301 Mission site was overlain by the margins of San Francisco Bay. As a result, the site is underlain by bay deposits, including silts, sands and clays, extending to several hundred feet below surface. These deposits include thick layers of saturated clay soils, known locally as Bay Mud.

Construction of the project required temporary dewatering of the site to permit excavation for subgrade levels and foundations. As the site was dewatered, the effective weight of soils in the dewatered zones increased, resulting in an increase in effective stress on Bay Mud deposits below and consolidation of these soils. This consolidation in turn resulted in total and differential settlement across the site. As construction proceeded, and reached grade, dewatering efforts were terminated, restoring buoyancy to site soils, allowing settlement to stabilize. However, shortly thereafter, construction initiated on a series of projects located to the north and south of 301 Mission. Of most note is the construction of the Transbay Transit Center and an associated train tunnel that extends east to west directly along the 301 Mission site's south perimeter. Construction of the train tunnel entailed deep excavation and accompanied dewatering of this adjacent site. These operations resulted in further total and differential settlements of the 301 Mission building. By 2013, these settlements had become noticeable to residents of the structure as evidenced through cracking in concrete elements of the building as well as noticeable unevenness in surrounding paving and slab-on-grade.

Millennium Partners retained Simpson Gumpertz & Heger Inc. (SGH) to provide an independent evaluation of the effects of these settlements on the building's earthquake resistance.

1.2 Objective

The objective of this study is to determine the effect of differential settlement experienced by the 301 Mission Street building on its structural integrity and in particular, its earthquake resistance. Specifically, we evaluate the extent, if any, that site settlements have reduced the building's earthquake resistance and whether structural reinforcement of the building is needed to restore its resistance to original design levels.

1.3 Scope of Work

Our study included the following specific tasks:

- Reviewed available documentation for the original building design including project geotechnical reports, structural drawings and structural calculations.
- Reviewed post-construction survey reports indicating the progression and profile of settlement over time.
- Conducted site visits to the building to observe its condition and note evidence of structural distress or damage.
- Performed detailed linear and nonlinear structural analyses to assess the effect of settlement on the building and its foundations.
- Prepared this report documenting the findings of our study.

1.4 Project Description

The Millennium Tower, consists of a fifty-eight-story, 627 ft tall, reinforced concrete tower and adjacent podium. The Podium structure is further divided into a three-story low-rise and a twelve-story mid-rise. A seismic joint separates the Tower and Podium. Figure 1 is a plot plan for the project indicating the location of the building and surrounding development.

The tower is comprised of flat post-tensioned concrete slabs supported by reinforced concrete frames (beams and columns) and a centrally located tube comprising reinforced concrete load-bearing walls. It is supported on a single, continuous 10 ft thick pile cap over pre-cast concrete piles. The basement contains a PG&E vault supported on a 3 ft thick slab cantilevered off of the pile cap. The Tower's lateral (wind and earthquake) resistance is a dual system consisting of concrete special moment frames around the perimeter and a concrete shear wall core with outriggers. Figure 2 is a typical tower floor plate, Figure 3 is a sectional elevation through the project looking to the north and showing the outriggers, and, Figure 4 is a foundation level plan.

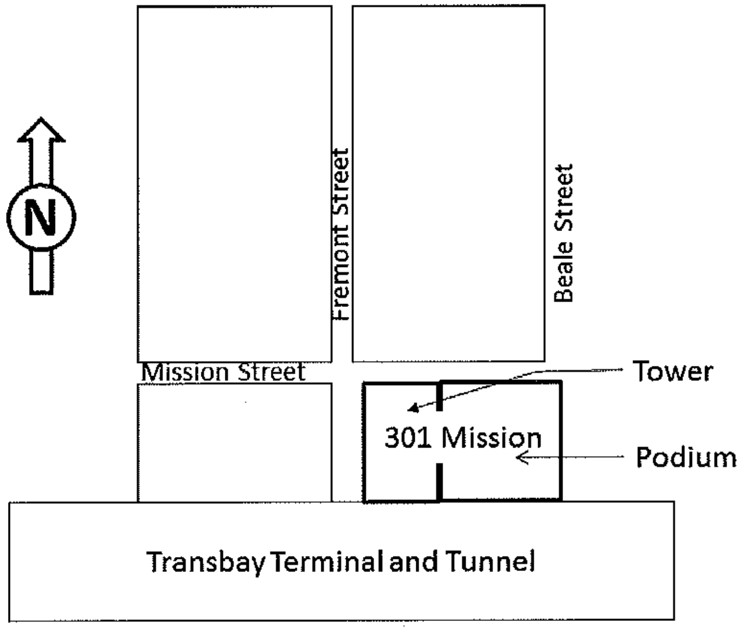


Figure 1 - Plot Plan 301 Mission Street

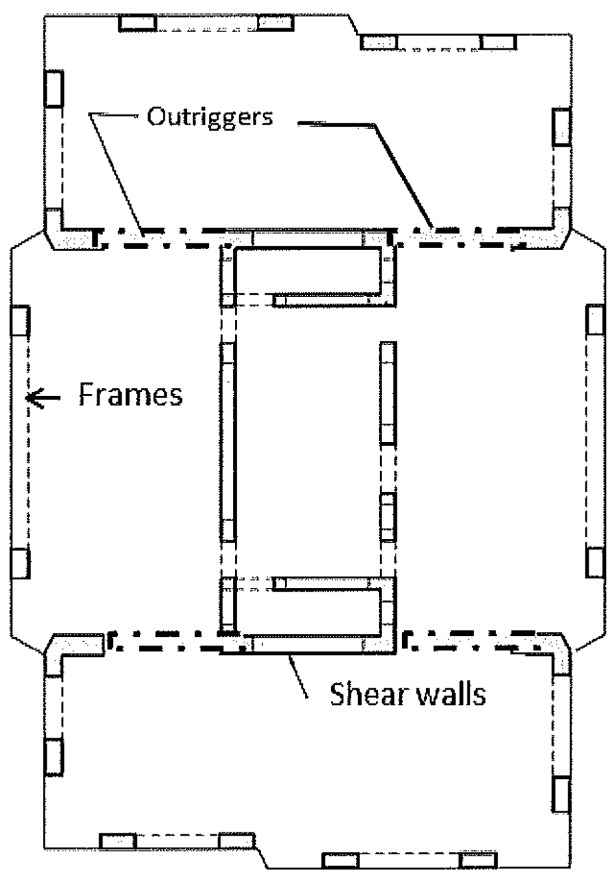


Figure 2 - Tower Framing Plan

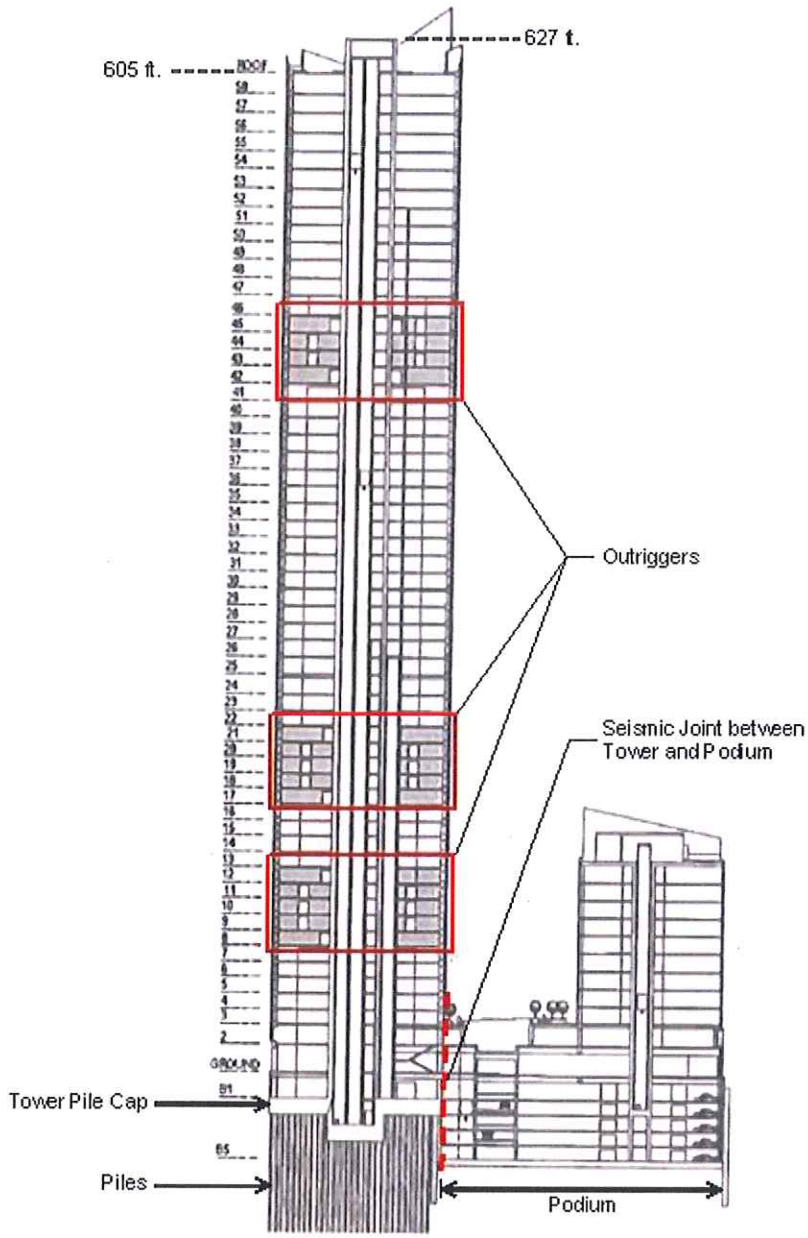


Figure 3 - Sectional Elevation Looking to North

The Podium contains five sub-grade levels supported on a mat foundation. Hinge slabs are used to connect the Tower and Podium in certain locations and allow differential movement between the two structures. Tie down anchors located under the low-rise portion of the Podium are used to resist hydraulic uplift pressure. The lateral system for the mid-rise consists solely of concrete shear walls.

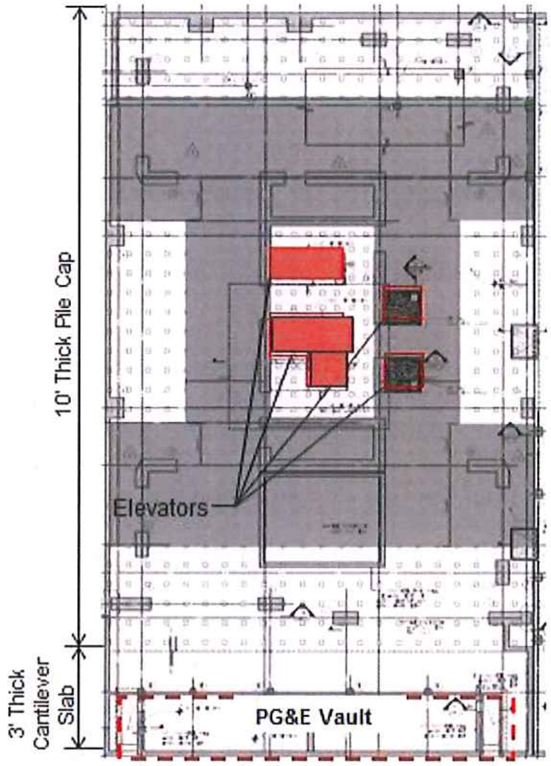


Figure 4 Foundation Plan

1.5 Relevant Parties

The following are principal parties to the project:

- Millennium Partners – the project developer.
- Handel Architects – the project architect.
- DeSimone Consulting Engineers – the project structural engineer.
- Treadwell & Rollo (aka Langan/Treadwell & Rollo) – the project geotechnical engineer.
- Transbay Joint Powers Authoirty (TJPA) – government agency developing the Transbay Terminal and associated train tunnel.
- Webcor Concrete- the project General Contractor.
- Arup – the engineering firm retained by TJPA to monitor settlement and lateral movement of buildings located around the Transbay Terminal and tunnel during construction.

2. SOURCES OF INFORMATION

This section summarizes sources of information for our study.

2.1 Treadwell & Rollo

We reviewed the following material prepared by Treadwell & Rollo:

1. Report: Revised Geotechnical Investigation, 301 Mission Street, San Francisco, dated 13 January 2005.
2. Memorandum to Steve Patterson of Millennium Partners, dated 24 February 2006.
3. Letter to Steve Patterson of Millennium Partners, dated 2 May 2006.
4. Letter to Raymond Lui of the City of San Francisco, dated 25 February 2009.
5. Memorandum to Steve Hood of Millennium Partners, dated 22 February 2012.
6. Memorandum to Steve Hood of Millennium Partners, dated 27 April 2012.
7. Memorandum to Steve Hood of Millennium Partners, dated 5 June 2012.
8. Memorandum to Steve Hood of Millennium Partners, dated 10 December 2012.
9. Memorandum to Steve Hood of Millennium Partners, dated 21 June 2013.
10. Memorandum to Steve Hood of Millennium Partners, dated 25 July 2013.
11. Memorandum to Steve Hood of Millennium Partners, dated 19 December 2013.
12. Memorandum to Steve Hood of Millennium Partners, dated 24 February 2014.
13. Memorandum to Steve Hood of Millennium Partners, dated 18 March 2014.
14. Memorandum to Steve Hood of Millennium Partners, dated 28 March 2014.
15. Memorandum to Steve Hood of millennium Partners, dated 3 October 2014.

In addition, we had a personal communication with Mr. Ramin Goelesorkhi of Treadwell & Roll.

The geotechnical report (1) indicates that the site is generally underlain by approximately 20 ft of gravelly sand and sandy gravel, with inter-bedded debris consisting of brick concrete and timber from older building construction on the site. Beneath this layer is approximately a 20 ft thick layer of Marine Deposits dating from the era when the site was under the waters of San Francisco Bay. These marine deposits consist of clay, sandy clay and clayey sands. Beneath the marine deposits are a layer of dense sands extending approximately 20 ft thick. Underlying the sands is a layer of clay and sandy clay, known as Old Bay Clays, that is at least 100 ft thick.

The geotechnical report recommends founding of the tower on driven piles extending to the dense sand layer. The report projects that settlements on the order of 4 to 6 in. could be anticipated resulting from consolidation of the Old Bay Clays that deeply underlie the site. Consolidation is a process by which clay soils are densified by the application of pressure such that water is "squeezed" out of the clay allowing it to compress. In the case of the 301 Mission Street project, consolidation is a result of pressures imposed on the Old Bay Clays by two effects: 1) temporary dewatering of the upper site soils to permit basement construction; and 2) the weight of the new construction itself.

In periodic memoranda to Mr. Steven Hood (5 – 15) Treadwell & Rollo document their evaluation and observations based on settlement reports for the site prepared by Arup under contract to TJPA. Treadwell & Rollo note that building settlement can be differentiated as occurring in four stages:

1. 30 April 2009 to 3 May 2010.
2. 3 May 2010 to 24 March 2011.
3. 24 March 2011 to 11 April 2014.
4. 11 April 204 to August 5 2014.

These stages of settlement are characterized by different rates of settlement and also by differences in the relative magnitude of settlement at different points on the site. The most significant rates of settlement occurred during the third stage during which settlement occurred at a rate of approximately 1 inch per year. During Stage 4, this has slowed to a rate approximately half this amount.

Treadwell & Rollo states that the larger than anticipated settlements are a result of:

1. An extended period of site dewatering during construction resulting in additional consolidation of the underlying soils.
2. Additional dewatering of the site, associated with the adjacent Transbay Terminal and train box construction.
3. Displacement of soils beneath the 301 Mission site towards the adjacent Transbay/train box construction, associated with the adjacent excavation and removal of piles from previous construction

As of the 3 October 2014 report, Treadwell & Rollo notes that the settlement rate has reduced significantly. Treadwell & Rollo attributes this to the progress of the adjacent construction and suggests that as the adjacent construction completes, ground elevations should stabilize.

2.2 Arup

We reviewed the following materials prepared by Arup:

1. Inclinometer Location Plan, March 2012.

2. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of Settlement Surveys at the 301 Mission Property, 12 March 2010.

3. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: December 2010 Settlement Surveys at the 301 Mission Property, 5 January 2011.

4. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: February 2011 Settlement Surveys at the 301 Mission Property, 24 February 2011.

5. Letter to Mr. Brian Dykes (TJPA) re Transbay Transit Center, Additional 301 Mission Instrumentation Installation, 28 March 2011.

6. Memorandum to Mr. Brian Dykes (TJPA) re: Transbay Transit Center – Pile Extraction Test Program – Measured Ground Deformations to Date, 8 April 2011.

7. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: March 2011 Settlement Surveys at the 301 Mission Property, 12 April 2011.

8. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Inclinometer Response to Productcion Pile Pulling in Zone 4, 9 May 2011.

9. Memorandum to Brian Dykes (TJPA) re: Results of Crack Guage Survey at 301 Mission, December 9, 2010; 16 May 2011.

10. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: May 2011 Settlement Surveys at the 301 Mission Property, 24 June 2011.

11. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Final Inclinometer Response to Production Pile Pulling in Zone 4.

12. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: July 2011 Settlement Surveys at the 301 Mission Property, 26 August 2011.

13. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: September 2011 Settlement Surveys at the 301 Mission Property, 10 October 2011.

14. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: October 2011 Settlement Surveys at the 301 Mission Property, 25 October 2011.

15. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: December 2011 Settlement Surveys at the 301 Mission Property, 23 December 2011.

16. Memorandum to Brian Dykes (TJPA) re: Results of Crack Guage Survey at 301 Mission, December 2011; 20 January 2012.

17. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of Tiltmeter Readings; 10 February 2012.

18. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Results of External Instruments Adjacent to 301 Mission Street, 7 March 2012.

19. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: March 2012 Settlement Surveys at the 301 Mission Property, 19 March 2012.

20. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: April 2012 Settlement Surveys at the 301 Mission Property, 27 April 2012.

21. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: June 2012 Settlement Survey at 301 Mission Property, 6 July 2012.

22. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Recent Manually Inclinometer Readings, 22 August 2012.

23. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of August 2012 Settlement Survey at 301 Mission Property, 22 August 2012.

24. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of October 2012 Settlement Survey at 301 Mission Property, 06 November 2012.

25. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center, Results of September 2012 Tape Extensometer Reading, 9 November 2012.

26. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Manually Read Inclinometer Update, 13 August 2012.

27. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of December 2012 Settlement Survey at 301 Mission Property, 17 January 2013.

28. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Manually Read Inclinometer Update, 14 February 2013.

29. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of February 2013 Settlement Survey at 301 Mission Property, 27 March 2013.

30. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center Internal 301 Mission Readings, 15 April 2013.

31. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center Internal 301 Mission Readings, 16 April 2013.

32. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Manually Read Inclinometer Update, 28 June 2013.

33. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of April 2013 Settlement Survey at 301 Mission Property, 28 June 2013.

34. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of June 2013 Settlement Survey at 301 Mission Property, 28 June 2013.

35. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of October 2013 Settlement Survey at 301 Mission Property, 13 November 2013.

36. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center – Manually Read Inclinometer Update, 10 December 2013.

37. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of December 2013 Settlement Survey at 301 Mission Property, 9 January 2014.

38. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of February 2014 Settlement Survey at 301 Mission Property, 28 February 2014.

39. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of April 2014 Settlement Survey at 301 Mission Property, 7 May 2014.

40. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of June 2014 Settlement Survey at 301 Mission Property, 27 June 2014.

41. Memorandum to Brian Dykes (TJPA) re: Transbay Transit Center: Results of August 2014 Settlement Survey at 301 Mission Property, 4 September 2014.

In this series of memoranda, Arup documents continuing settlement of the 301 Mission building as measured by a series of inclinometers, and manually surveyed benchmark locations in and around the building. Arup documented these recordings in the form of a series of inferred settlement contours with settlements measured relative to a fixed datum. Figure 5 is a plot of the settlement contours shown by Arup based on their April 2014 (39) measurements.

Review of these contours differential settlement across the tower footprint of approximately 6 in. with the most severe settlement occurring just to the west of the north end of the central core. Figure 6, also extracted from Arup's April 2014 measurement memo shows this data for the tower only, in the form of differential settlements. Settlements beneath the mid-rise podium structure are more uniform and exhibit differential settlement of approximately 2 in. across the foot print.

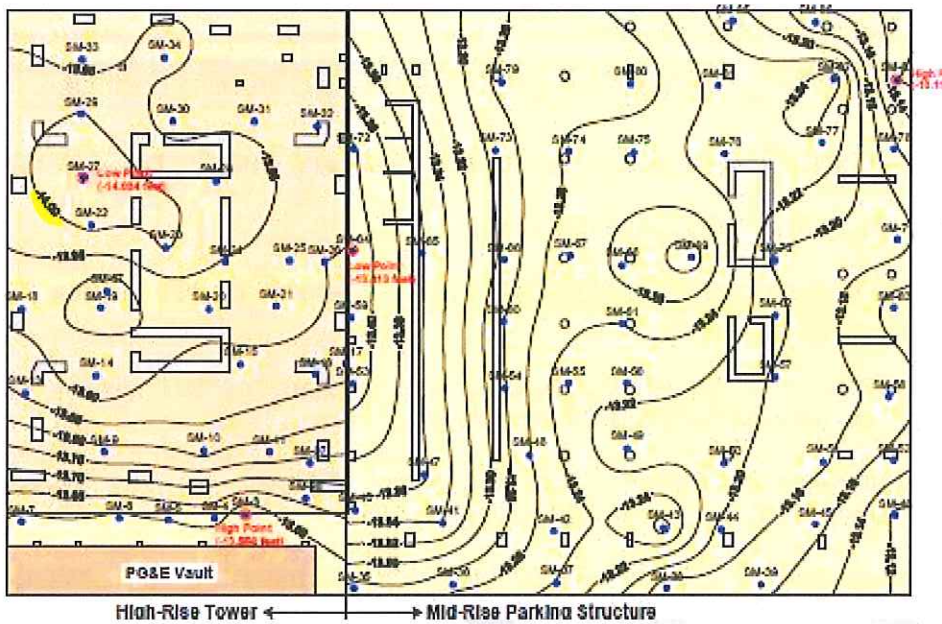


Figure 5 - Settlement Contours, April 2014 (Arup)

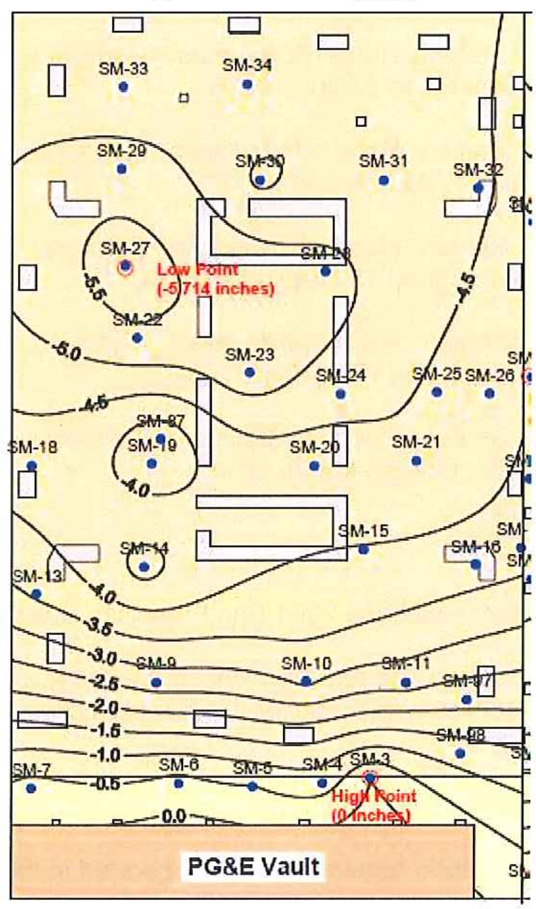


Figure 6 - Differential Settlement Contours Tower – April 2014 (Arup)

2.3 DeSimone Consulting Engineers

We reviewed the following materials provided by DeSimone Consulting Engineers:

1. Structural Drawings: 301 Mission Street (109 sheets) dated 30 December 2005.

2. ETABs Model 4069-20060313-315-CD.

3. Letter to Raymond Lui (City of S.F.) re: Settlement of 301 Mission Tower, dated 25 February 2009.

4. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – May Settlement Evaluation, dated 6 June 2012.

5. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 28 November 2012.

6. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 14 December 2012.

7. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 5 April 2013.

8. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 24 June 2013.

9. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 23 July 2013.

10. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 13 December 2013.

11. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 21 January 2014.

12. Memorandum to Steven Hood (Millennium Partners) re: 301 Mission Street – Settlement Evaluation, dated 6 March 2014.

The structural drawings indicate:

1. Design is in accordance with the 2001 San Francisco Building Code (SFBC).

2. The designated seismic force-resisting system for the tower consists of a dual system comprising reinforced concrete special moment-resisting frames and concrete shear walls with outriggers

3. Design seismic forces are computed using the modal response spectrum procedure.

4. Enhancements to the code requirements incorporated in the design include:

    - Outrigger link beams are designed for amplified seismic forces including consideration of over-strength.

- Columns supporting the outriggers and the attachment of the outriggers to the shear walls are proportioned to resist the expected strength of the outriggers while remaining elastic.
- The pile cap is designed to remain elastic when subjected to the capacities of the outrigger columns and the maximum anticipated moment capacity of the concrete shear walls.

2.4 Handel Architects

We reviewed an 18 February 2009 letter from Handel to DeSimone Consulting in which Handel discusses modifications made to the design to accommodate larger than anticipated settlement and differential settlement. These modifications include:

- Installing utility lines with flexible connections where they cross the seismic joint between the tower and mid-rise podium.
- Installing handrails at hinge slabs between the Podium and Tower to account for the increased slope due to settlement.
- Re-routing utilities.
- Re-designing seismic joint covers at walls, ceilings, and floors.
- Raising interior floor levels or installing new trench drains to prevent water drainage towards entry and exit doors.

3. FIELD INVESTIGATIONS

On 1 April 2014, Anindya Dutta, S.E. and Kyle Douglas, P.E. of SGH visited the building to observe its condition. Observation was limited to areas where the concrete structure is not covered by architectural finishes and therefore, could be directly observed. Specifically, this included the mechanical and electrical rooms in the tower basement and the stair wells at the eight through twentieth floors.

We did not observe any visible cracks in the pile cap. There was also very little cracking in the shear walls. Only the basement shear wall in the north stairwell had any visible cracking (Figure 7). We also observed cracking in the first floor slab in the northwest corner of the basement (Figure 8).

The most notable cracking observed is in the moment frames and basement wall in the south and southeast area of the basement. We observed diagonal cracks near the beam-column connections of both southern moment frames (Figure 9 - Figure 11) as well as cracking at the support of a gravity beam where it connected with the southeast moment frame (Figure 12). Finally, we noted significant cracking in the southeast basement wall near the moment frame (Figure 13).

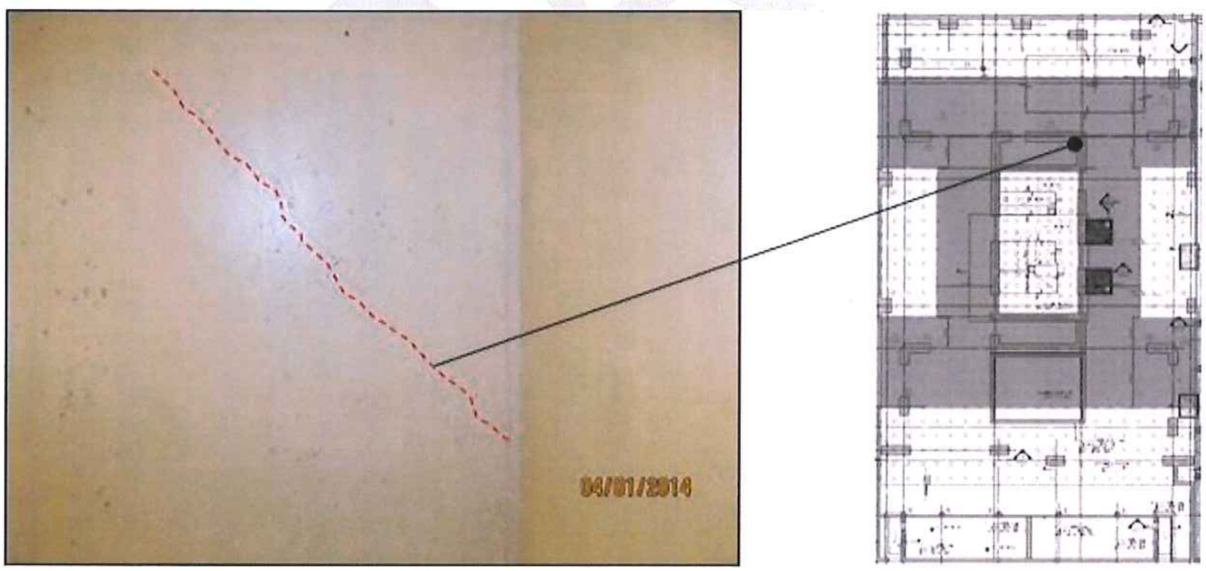


Figure 7 - Shear wall crack in tower basement

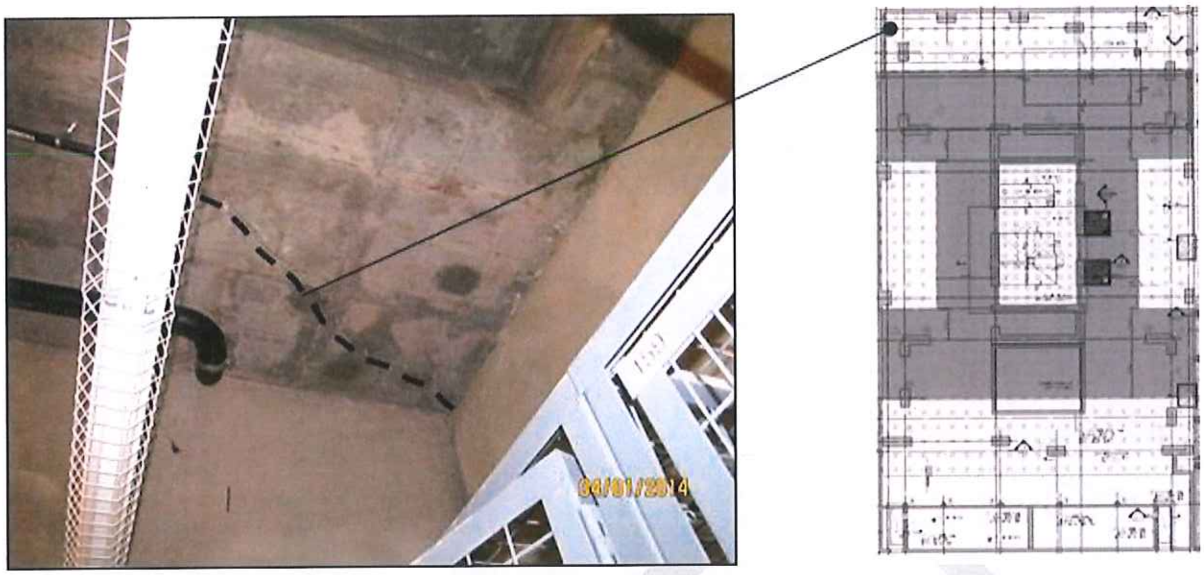


Figure 8 - Crack in underside of 1st floor slab

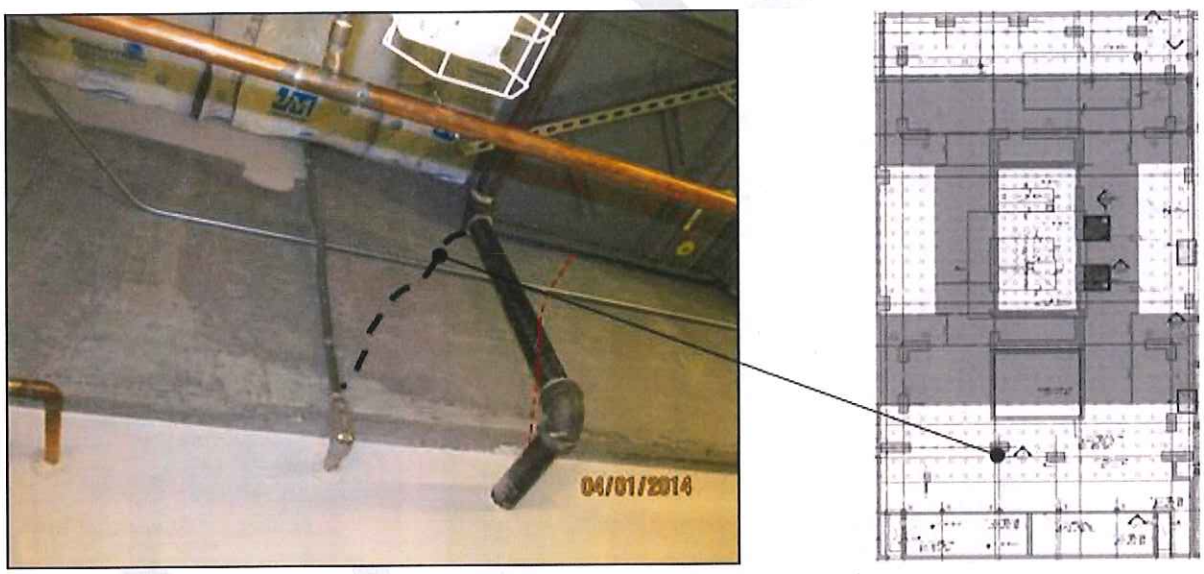


Figure 9 - Diagonal crack in moment frame beam along Line G.8 near beam-column joint

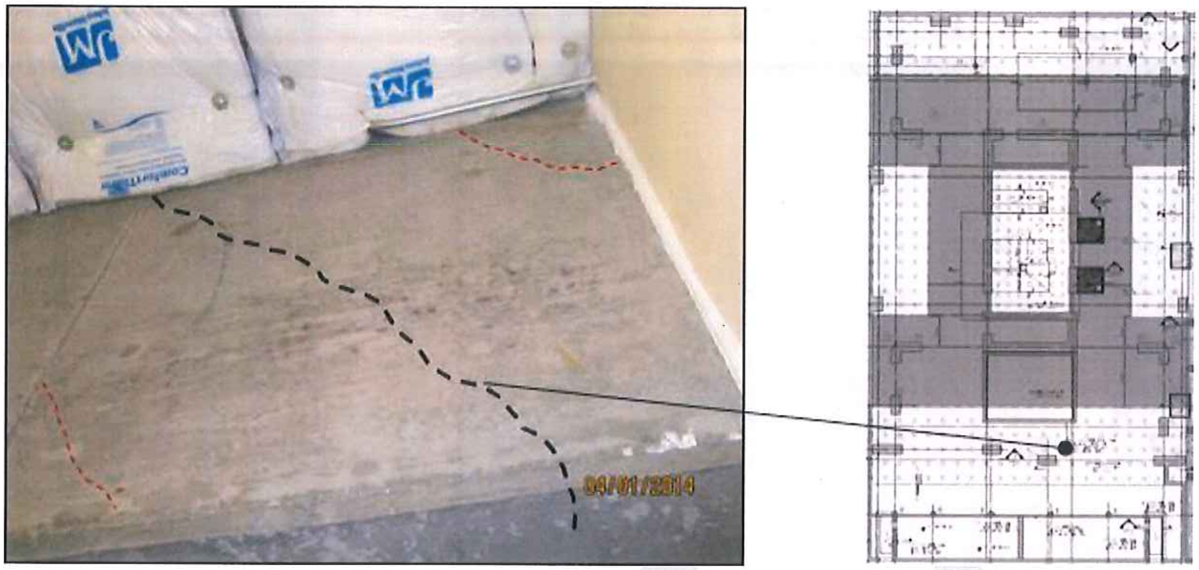


Figure 10 - Diagonal crack in moment frame beam, Line H near beam-column connection

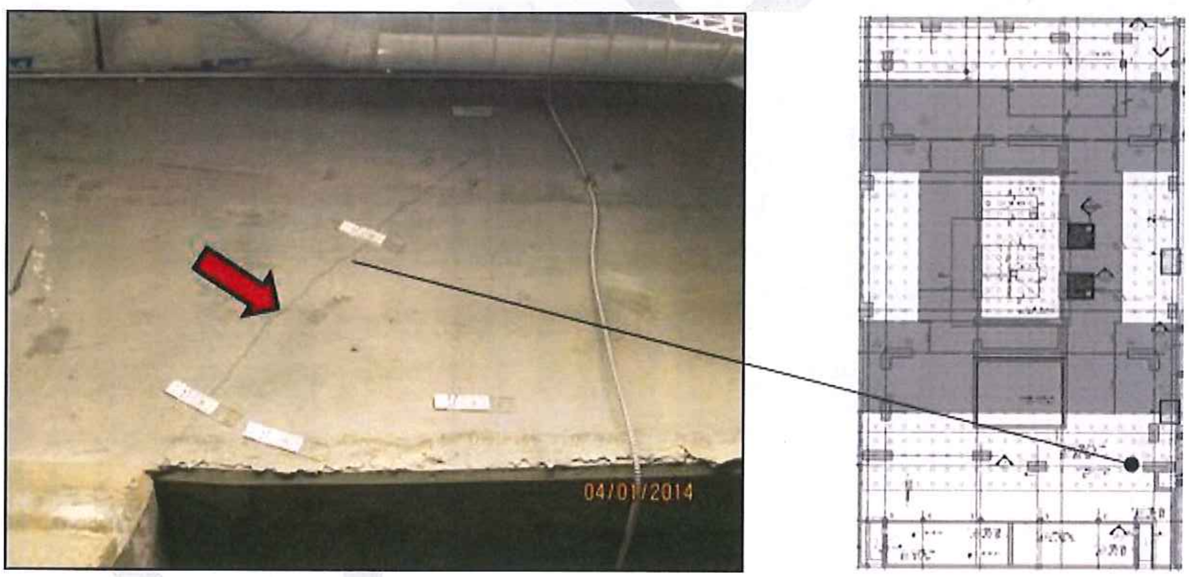


Figure 11 - Shear crack at beam-column joint, Line H

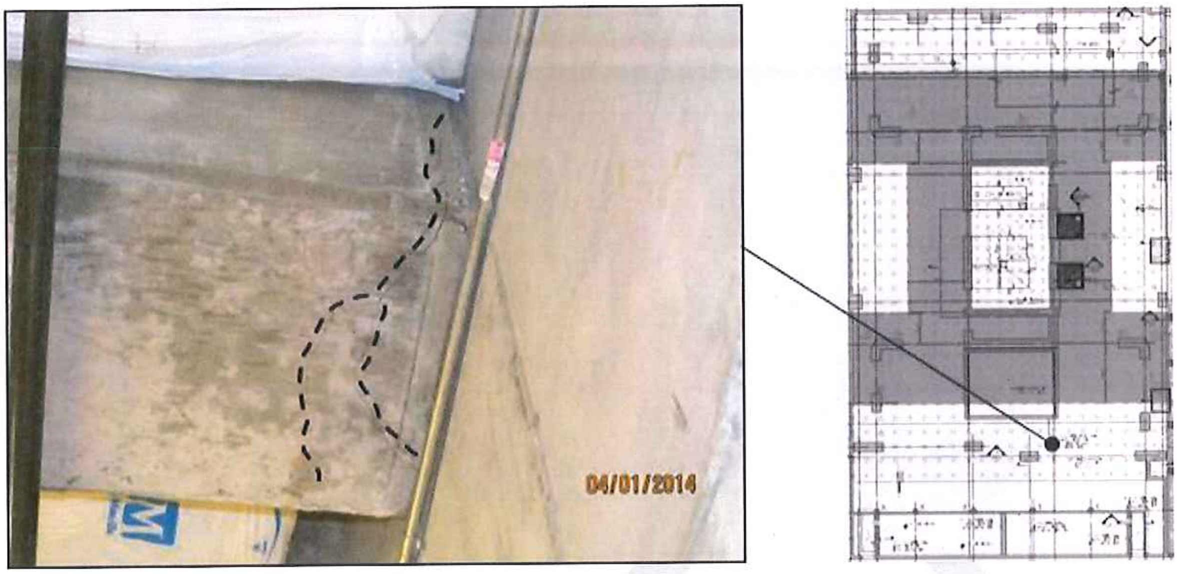


Figure 12 - Cracks in gravity beam framing to moment frame

Figure 13 - Cracks in basement wall near moment frame

On 6 May 2014, Anindya Dutta, S.E. and Kyle Douglas, P.E. of SGH rode on top of elevator S-1 to observe the condition of one of the core shear walls. Figure 14 shows the location of the elevator and the shear wall (labeled Shear Wall A) observed.

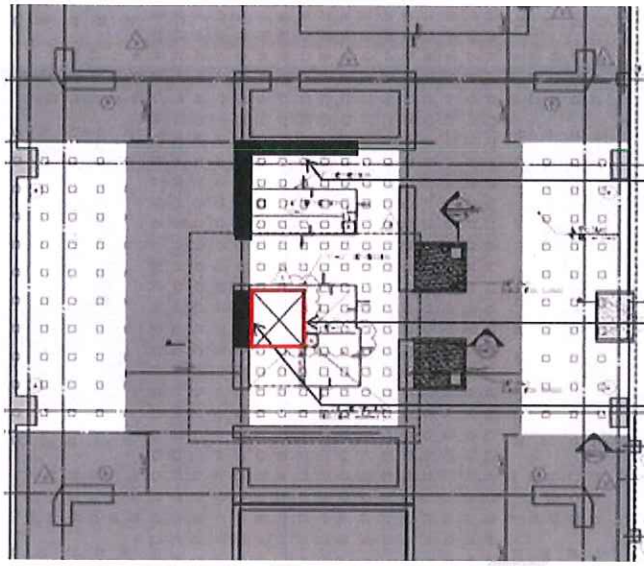
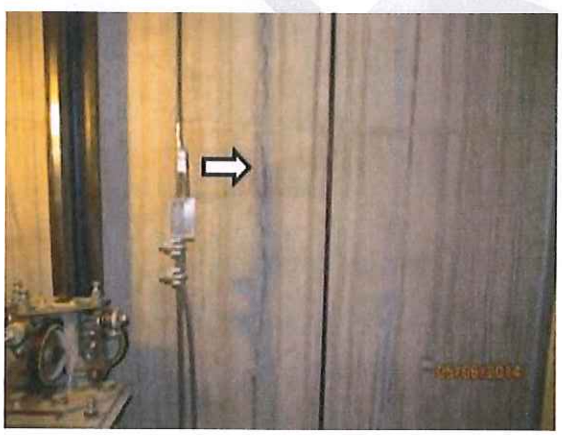


Figure 14 - Location of shear wall observed from elevator cab

Generally, the shear wall is in good condition. We observed minor cracks which appeared to be shrinkage cracks, and cracks located at penetrations in the wall. Figure 15 shows a typical crack. Figure 15a shows the total length of the crack and 15(b) a close-up view of the crack at a penetration. Note that the dark line in Figure 15b is not the crack itself, but an indicator marking placed next to the crack for identification purposes. We were also able to observe a few link beams framing into the shear wall. The link beam connections did not show any signs of distress or cracking. Figure 16 shows a typical link beam connection.



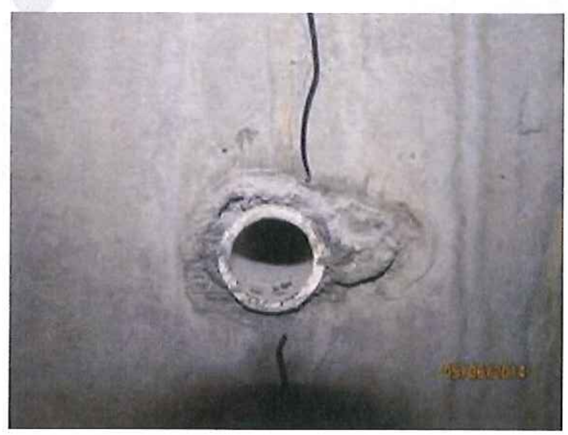


Figure 15 - Cracks observed in shear wall at elevator shift (a) from a distance and (b) at a penetration

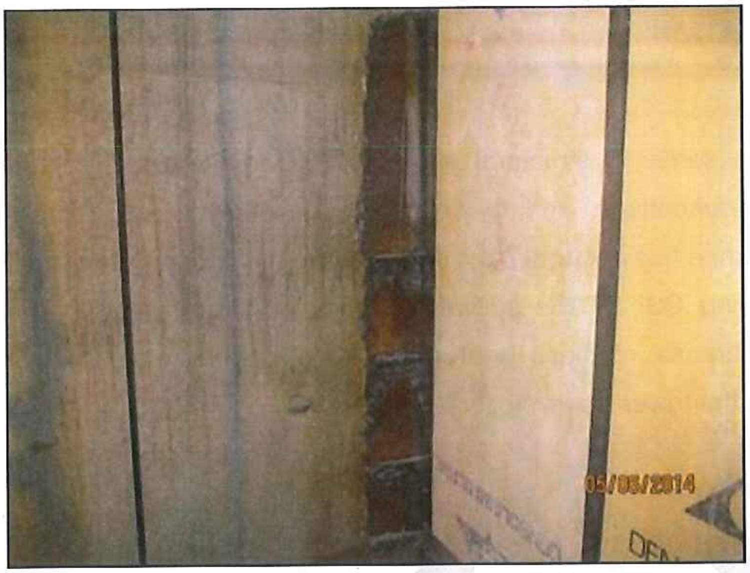


Figure 16 - Link beam to shear wall connection

We also investigated the elevator pit shear walls adjacent to the north stairwell (labeled Shear Wall B in Figure 14). We observed multiple diagonal cracks in the east-west wall at the northwest corner of the elevator pit (Figure 17). Note that this is the opposite face of the shear wall in which we found cracking during the basement field investigation (Figure 7).

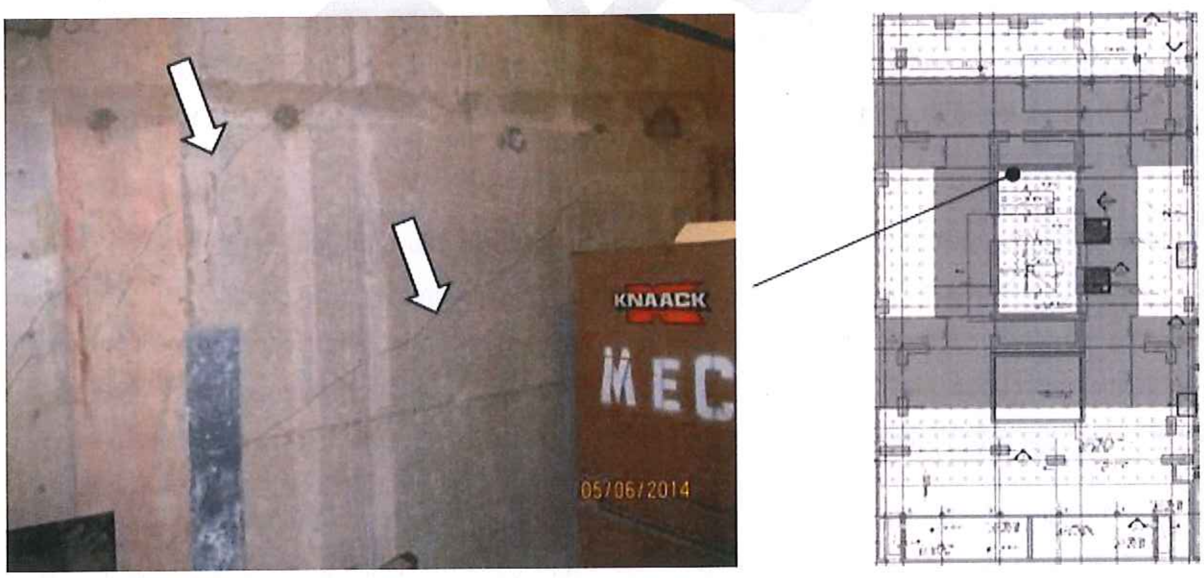


Figure 17 - Diagonal cracking shear wall adjacent to stair well

4. CALCULATIONS

4.1 General

SGH performed a series of structural analyses to evaluate the effect of settlement on the building and its foundations, and to determine if significant degradation in the building's earthquake resistance has occurred as a result of the settlement. Specifically, we performed a linear analysis using CSI ETABs software and a nonlinear analysis using CSI PERFORM software. The purpose of these analyses was to determine what effect, if any, building settlement had on the tower's seismic force resistance.

4.2 Linear Analysis

4.2.1 Introduction

Linear structural analysis is a method of computing the deformations and stresses that occur in a structure when subjected to applied loads. Loading can consist of externally applied forces, such as wind load, externally imposed deformations such as foundation settlement, or a combination of these. Linear analysis uses a mathematical model of the structure that simulates the behavior of a series of interconnected springs. Each spring has defined stiffness, which is the amount of force necessary to produce a deformation of unit magnitude in the element. A fundamental assumption of linear analysis is that the stiffness of the various springs remains unchanged regardless of loading magnitude. Thus, under linear analysis it is assumed that if a structure is loaded with 1,000 lbs and deflects 1 in., when loaded with 10,000 lbs it will deflect 10 in. and 100,000 lbs will deflect 100 in.

Linear analysis methods provide reasonable characterization of structural behavior under moderate levels of loading that does not cause structural damage, e.g., cracking of concrete or yielding of steel. Linear analysis does not provide reasonable characterization of structural behavior under load intensities that produce damage, because such damage tends to alter the stiffness of damaged elements, a behavior that linear analysis is unable to simulate.

Building codes specify the intensity of various load types, such as wind load, occupancy-related (live) load, and earthquake load that engineers must design buildings to resist. Most of these loads, such as wind and live loads are anticipated to occur many times during a building's life. Consequently, the building codes specify that structures be designed such that they are capable of resisting design wind and live loads without damage, so that building owners do not have to repair the structure each time it is loaded. Linear analysis is an ideal means of evaluating the force, stress and deformation demands under such loads.

Unlike wind loads and live loads, damaging earthquakes are rare events having mean recurrence intervals ranging from hundreds to perhaps thousands of years. Most buildings have useful lives less than 100 years and as a result will never experience a major damaging earthquake. Therefore, for economic reasons, building codes do not seek to avoid damage under design earthquake shaking, but rather, seek only to avoid structural collapse, or other severe damage that could endanger the safety of building occupants. It is expected that when buildings designed to the building code are subjected to a design earthquake they will experience substantial damage and as a result significant reductions in the stiffness of the beams, columns, walls and other elements that comprise them. Linear analysis is not capable of providing accurate evaluation of a building's performance under such loading.

Despite the inherent inaccuracy of linear analysis in predicting the behavior of buildings in strong earthquake shaking, the building code requirements for earthquake resistance assume the use of linear analysis in design and most buildings designed for earthquake resistance today, including the 301 Mission building, are designed using linear analysis techniques. The reasons for this are largely historic. Until approximately twenty years ago, linear analysis techniques were the only ones available to engineers. Observation of the performance in real earthquakes of buildings designed using linear analysis techniques demonstrated that with proper controls on the design process, buildings designed using linear analysis could provide acceptable earthquake behavior.

The building code procedures for earthquake-resistant design using linear analysis techniques require the following:

1. That earthquake resistance be provided by use of one of several recognized structural systems that have been proven to provide satisfactory performance in past earthquakes.

2. The details of construction including the quality of materials and workmanship and the way individual members are proportioned and joined together must conform to strict rules prescribed by the building code.

3. The building must have adequate strength to resist specified seismic design forces in combination with the structure's self-weight (dead load), live and other loads.

4. The building must have adequate linear stiffness to resist specified seismic design forces without excessive lateral deflection, sometimes also called "drift."

The specified design seismic forces for a building are based on the building's location relative to potential earthquake sources; the building's weight; the building's dynamic properties; and the type of structural system used to resist seismic forces. Depending on the type of structural system used, the specified design seismic forces can be as little as 1/8 of the forces that the structure would actually experience in a design earthquake, if it were able to resist such forces without damage. This anticipates that the structure will experience extensive damage.

Although linear analysis is not capable of accurately predicting a building's behavior in response to design earthquake shaking, or severe settlement, we performed a linear analysis of the structure to determine the stresses imposed on various elements of the structure's seismic force-resisting system by the settlement assuming that the structure is undamaged by these effects. This enabled us to determine utilization ratios, which indicate how much of the structure's available strength is mobilized as a result of the settlements. This provides a crude understanding as to whether significant reduction in strength available to resist earthquake shaking has occurred. We note that this represents a crude first order approach to determine if settlements have compromised the building's earthquake resistance.

4.2.2 Modeling

We performed our linear analysis using ETABs 2013 Version 1.4. ETABs is a structural analysis software developed and marketed by Computers & Structures Inc. of Berkeley, CA. This software was specifically developed to perform structural analysis and building code acceptance checking of building structures and is commonly used by California engineers to design building structures for earthquake resistance.

We performed our analysis using a model originally developed by DeSimone Consulting Engineers during original building design. Figure 18 shows the ETABs model, which includes the shear walls, columns and beams forming the moment-resisting frames, outriggers and foundation mat. Prior to using the model for our analyses, we verified that it accurately represented DeSimone's design, as depicted on the structural drawings. Where we found that the model did not match the design, we modified it to more properly reflect the design shown on the drawings. Specifically we made the following modifications:

- Adjusted the location of some moment frame columns.
- Updated element sizes for selected beams and columns forming the moment frames.
- Updated selected shear wall thicknesses.
- Updated concrete modulus of elasticity for some shear wall elements.
- Removed 2nd floor braces in moment frames at Lines G.8 and H

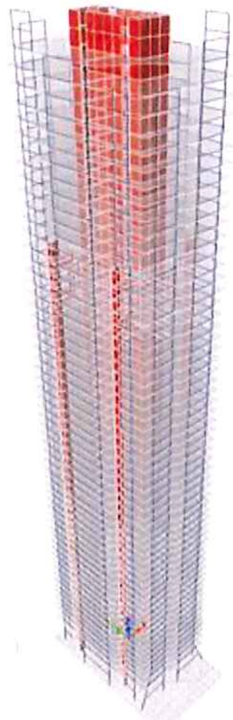


Figure 18 - Isometric view of ETABs Model

In addition to the above modifications, we also made a number of modifications necessary to enable the model to better predict response to building settlements. These included:

- Removed flexural modifiers for link beams.
- Removed gravity load carrying columns in the northwest and southwest corners of the model.
- Removed the rigid diaphragm designation for floor slabs, allowing their in-plane stiffness to be determined based on their properties.
- Adjusted the P-Delta definitions to incorporate the structure's self-weight and super-imposed dead load.
- Added the cantilever slab at the PG&E vault.

We re-constructed representation of the pile cap so that the nodes coincided with the location of settlement readings, columns, and shear walls. Figure 19 is a plan view showing the pile cap modeling and the locations of the settlement measurements.

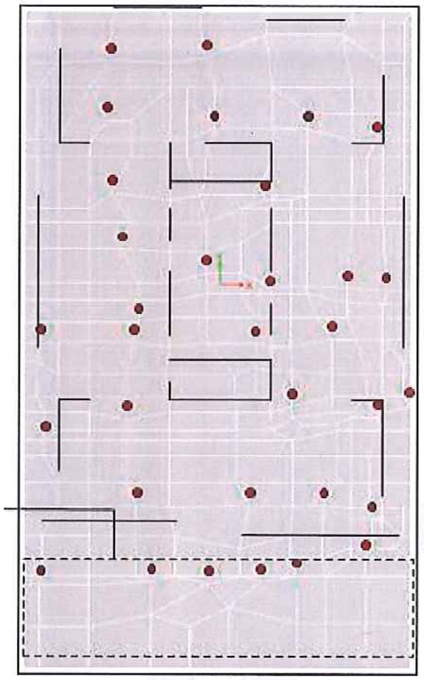


Figure 19 - Plan of Pile Cap Model

Finally, we modified (reduced) the stiffness of the floors, walls, pile cap, cantilever slab, and frame elements to account for cracking. Table 1 summarizes these modifiers.

Table 1: Property Modifiers used in the ETABS model

ELEMENT	PROPERTY	MODIFIER
Pile Cap and Cantilever Slab	Flexure	0.40
Walls	Axial, Flexure	0.60 (B-32) 0.85 (32-61)
Floors	Axial, Flexure	0.80
Columns	Axial Torsion, Flexure	0.70 (B-18), 0.60 (18-61) 0.50
Beams	Torsion, Flexure	0.50

4.2.3 Loading

To study the impact of settlement on the tower, we imposed the measured elevation profile reported by Arup in their 28 February 2014 report on the pile cap in our ETABS model. Arup reports elevations only at a discrete number of points. Therefore, the exact profile across the mat can only be estimated. To do this, we first first calculated the relative elevation at each settlement reading from the following equation:

$$\Delta_{relative} = H_{measured} - H_{max}$$

where: $\Delta_{relative}$ is the relative elevation at a given reading (inches); $H_{measured}$ is the measured elevation at a given reading (inches) reported by Arup and H_{max} is the maximum elevation for all readings (inches). Then, using the relative elevations at the points recorded by Arup we assumed that mat conforms to a smoothed curved surface having elevations matching the recorded values at the measurement points. We then used *Surfer 8* software to calculate the corresponding elevation of each nodal point in our mat model. Finally, we imposed these relative elevations as enforced displacements on the ETABs model. Figure 20 shows the calculated contours for the pile cap that we imposed on the model. Note that these contours match those reported by Arup in their 28 February 2014 report, extracted here as Figure 21.

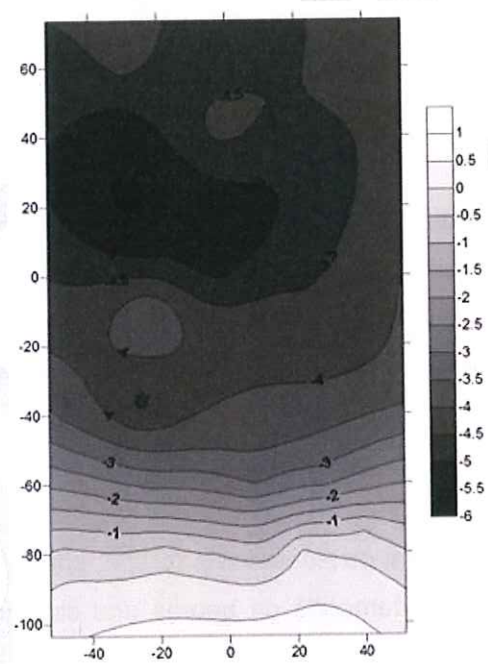


Figure 20 - Computed settlement contours imposed on model

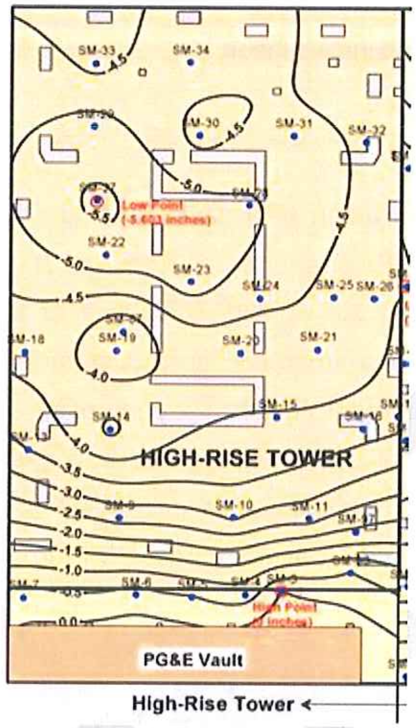


Figure 21 - Settlement contours reported by Arup (28 February 2014)

Our site observations of the core walls indicated that they were essentially uncracked, suggesting that the core has settled and rotated as a rigid body. Given the great stiffness of this tall concrete wall system, we believe such behavior is likely. Therefore, we modified the relative elevations along the core shear walls so that the profile of the pile cap was linear, representing this rigid body motion. This recognizes the stiffness of the shear walls and promotes rigid body rotation of these elements.

4.2.4 Results

We evaluated overall building lateral displacements (drifts), shear stresses on shear walls and outriggers, and axial and flexural demands on beams and columns forming moment frames. We discuss each of these below.

4.2.4.1 Lateral Drift

Figure 22 shows the tower's lateral displacement due to settlement as calculated by our linear analysis. Computed displacement at the roof is 2.7 in. west and 7.0 in. north. The elevation of the roof floor is 627 ft. These displacements therefore represent building lateral drifts of 0.04% and 0.10% of the building height, respectively, in the west and north directions. The present California Building Code permits computed lateral drift under earthquake shaking of 2% of building height. We conclude that settlement has utilized approximately 5% of the building's permissible drift capacity, which is a negligible amount.

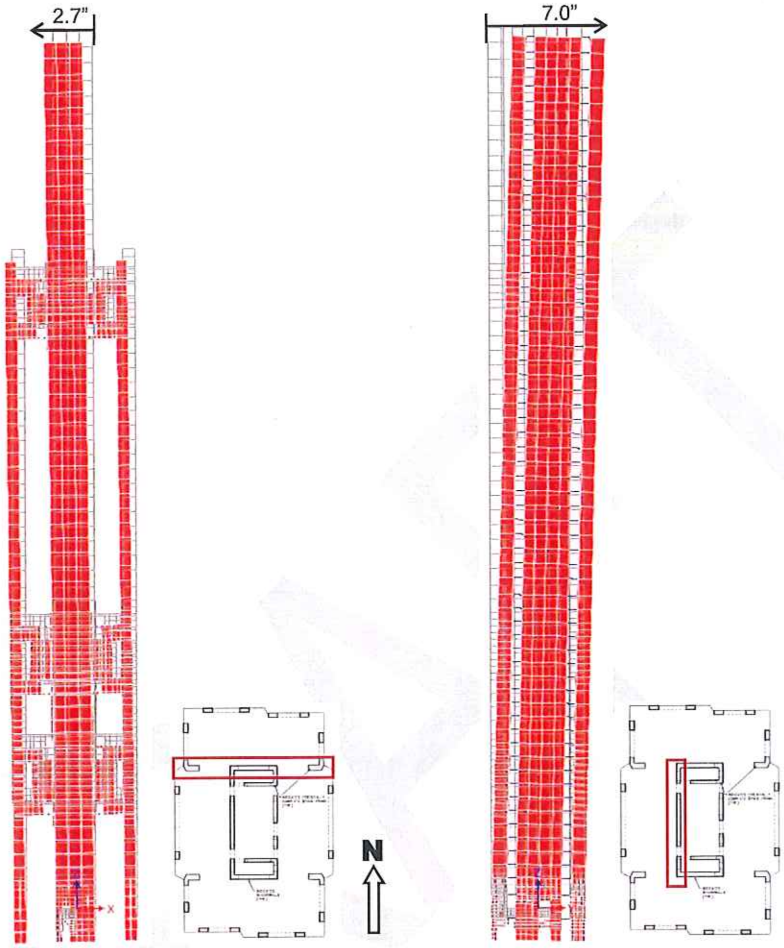


Figure 22 - Exaggerated drift profile computed for tower

4.2.4.2 Moment Frames

Utilization ratios in beams and columns forming moment frames are generally below 0.30 indicating that the effects of settlement on these elements are generally small. Only north-south oriented moment frames associated with outrigger columns, and shown in Figures 23 and 24 exhibit utilization ratios greater than 0.30. Notably, utilization ratios in these moment frames decrease with height in the structure, with the greatest utilization ratios at the lower levels of the tower. This is significant because in dual system structures, i.e. structures with seismic force-resisting systems composed of both shear walls and moment frames, the shear walls are most heavily stressed by earthquakes at the bottom of the structure and the moment frames at the top. This implies that where stresses due to settlement in the moment frames are high, stresses due to earthquake shaking are relatively small such that the effect of settlement on earthquake resistance is of limited importance.

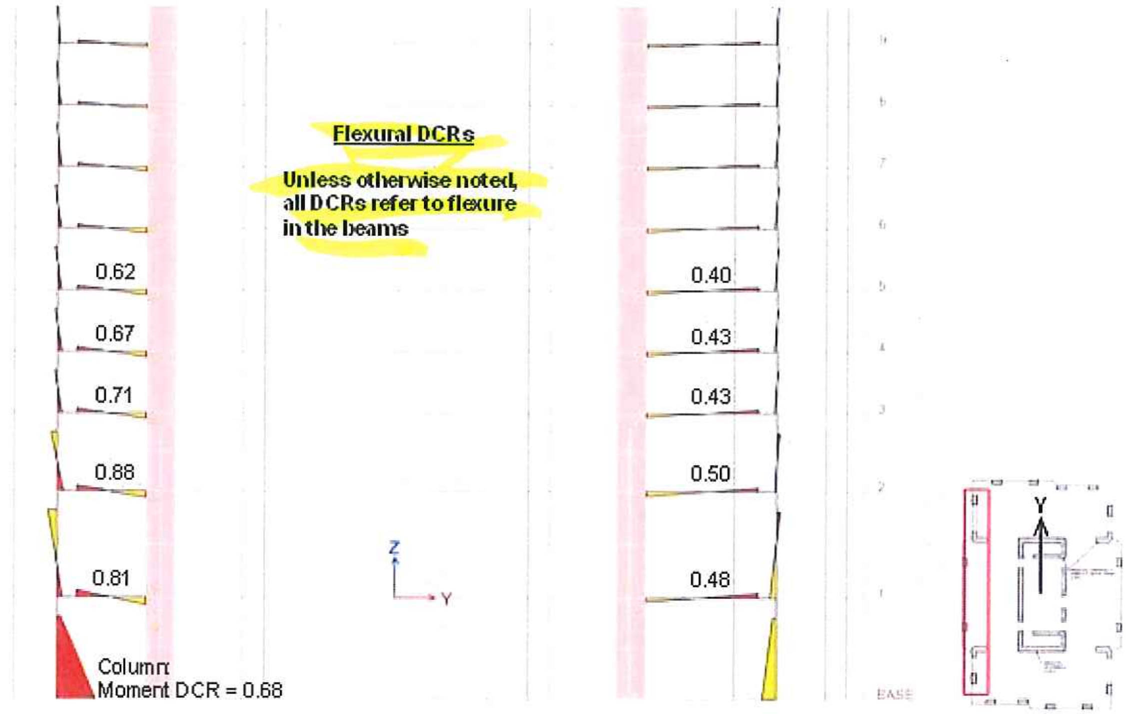


Figure 23 - Utilization (DCR) ratios in moment frames, Line 2 (west side)

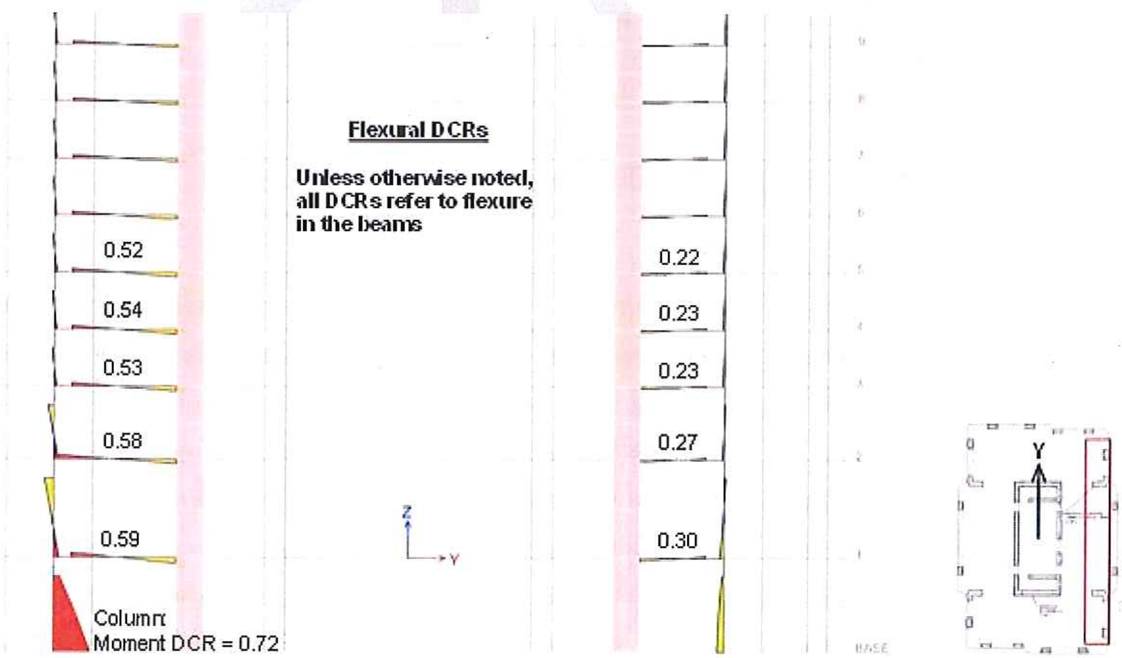


Figure 24 - Utilization ratios (DCRs) moment frames Line 11 (east side)

4.2.4.3 Shear Walls and Outriggers

Figure 25 and Figure 26 present maximum computed shear stresses in the shear walls and outriggers. Shear stresses are represented as a multiple of the square root of the specified concrete compressive strength $\sqrt{f'_c}$. Only shear stresses equal to or greater than $1.0\sqrt{f'_c}$ are presented. Wall and outrigger capacity is generally in excess of $8\sqrt{f'_c}$. The greatest shear stresses are in the southwest corner of the structure, as indicated by the spandrel in Figure 26, the high shear stresses in the west outrigger columns (Figure 25), and the high shear stresses in the southern outrigger (Figure 26).

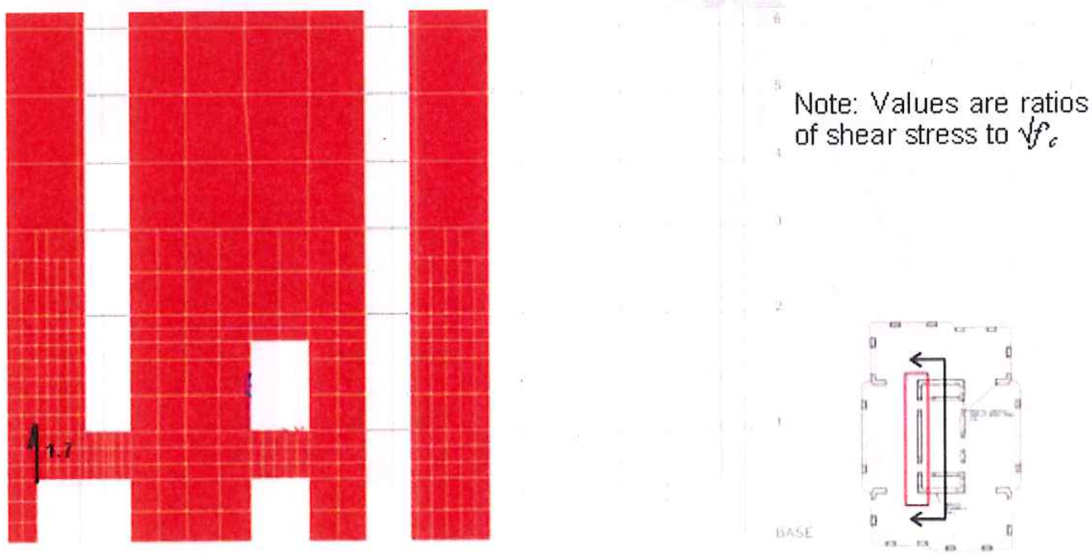


Figure 25 - Shear wall stress exceeding $\sqrt{f'_c}$, Line 4

The shear stresses in the outrigger beams shown in this analysis are significant. However, they do not account for potential flexural yielding of the beams, which would prevent overstress of these outriggers under earthquake loading. It is not possible to account for this effect using linear analyses.

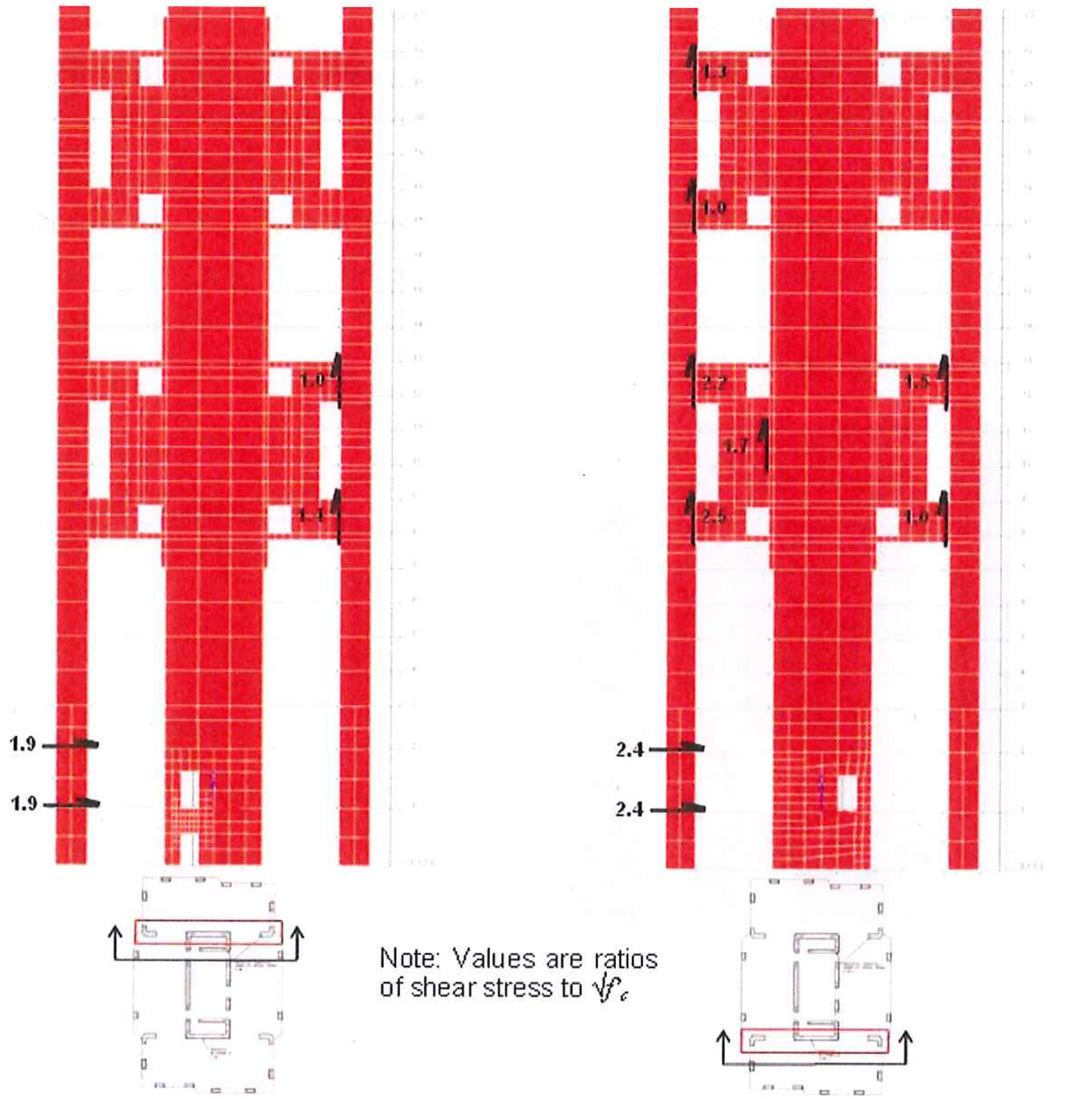
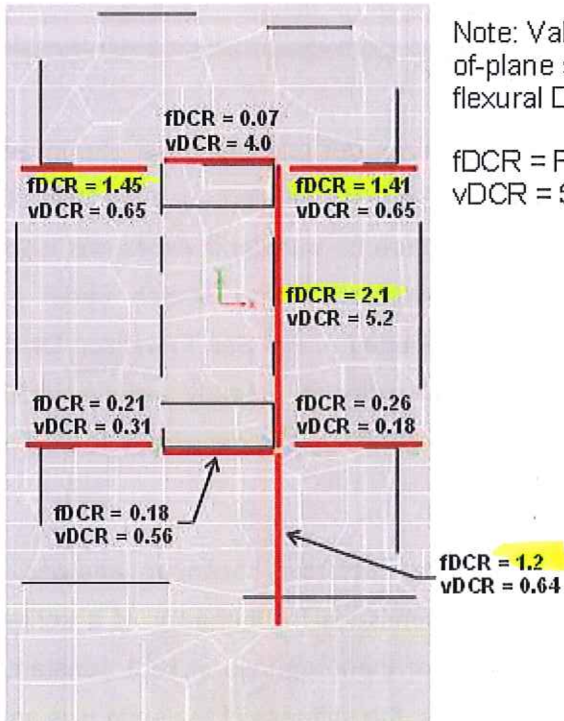


Figure 26 - Shear wall stress exceeding $\sqrt{f'_c}$, north and south core walls and outriggers

4.2.4.4 Pile Cap

The tower is founded on a thick reinforced concrete mat that is supported on a large number of precast concrete piles, driven into the ground. Figure 27 shows the utilization ratios we computed for the concrete mat in bending (fDCR) and shear (vDCR). In some locations adjacent to the shear walls these utilization ratios are substantially in excess of 1.0 suggesting that building settlement has resulted in significant yielding of the mat independent of earthquake loading.



Note: Values are out-of-plane shear and flexural DCRs

fDCR = Flexural DCR
vDCR = Shear DCR

Figure 27 - Shear and flexural utilization ratios in pile cap

4.2.5 Summary

Our linear analyses of the structure for the effects of settlement indicate the following:

- The lateral displacement of the tower due to settlement is very small relative to the height of the structure and a small fraction of that permitted under earthquake loading.

- Generally, settlement-related stresses in most moment frame elements are small indicating relatively little effect. In some few moment frame members, mostly located near the structure's base, utilization ratios are somewhat larger, indicating that the remaining capacity for seismic loads is somewhat reduced. However, these elements are generally located in regions where stresses associated with earthquake shaking are expected to be limited.

- The shear stress in the outriggers exceeds $2.0\sqrt{f'_c}$ in a few areas, also indicating relatively high stress levels due to settlement.

- The pile cap is highly stressed in several areas, with moment and shear utilization exceeding elastic capacity.

On the basis of these analyses, we determined that to accurately quantify the structure's earthquake resistance, given the settlement that has occurred, it was necessary to perform nonlinear analyses. The next section presents these analyses.

4.3 Nonlinear Analysis

4.3.1 Introduction

Modern building codes require that structures designed to resist strong earthquakes, like the Millennium Tower, incorporate design features intended to allow the structures to exhibit ductility. Ductility is the ability of a structure to withstand excessive loading without failure. When ductile concrete structures are overloaded they stretch under loading rather than breaking. The seismic design criteria contained in the San Francisco Building Code explicitly rely on the existence of this ductility. In order to properly assess whether a building has adequate ductility to resist design seismic shaking, it is necessary to perform nonlinear rather than linear analysis.

As with linear analysis, when engineers perform nonlinear analysis, they construct a mathematical model of the building that simulates the behavior of a series of interconnected springs. In linear analysis, the stiffness of these springs is kept constant, regardless of the magnitude of loading. In nonlinear analysis, the stiffness of the springs is adjusted as loading is applied to the structure, to account for the effects of cracking of concrete and yielding of reinforcing steel that occurs when structures are over loaded. This ability allows the analysis to represent the beneficial effects of ductility when a structure is subject to extreme loading and to determine whether such structures are able to resist this loading safely.

We performed our nonlinear analyses using the computer program Perform-3D V5.0.0, developed by Computers & Structures Incorporated of Berkeley, California. This software is commonly used by engineers in California designing buildings using performance-based approaches. Over the past ten years, most high-rise buildings constructed in California have been designed using these techniques and this particular software.

In our analyses we first constructed an analytical model of the structure, with nonlinear representation of the strength, stiffness and ductility of the various framing elements. Our assumptions as to these properties are based on data presented in ASCE 41-06 *Seismic Rehabilitation of Structures*, a national consensus standard referenced by the building code for nonlinear seismic analysis. Then we applied gravity loading to the model, representing the effects of the structure's self-weight (Dead Load) and that of its occupants (Live Load). We then imposed a settlement profile on the mat foundation, similar to that described under our linear analyses. Finally, we applied a suite of seven earthquake ground motions to the building to simulate its response to Maximum Considered Earthquake shaking. This is a very severe level of shaking, and for sites in the San Francisco Bay Area is expected to occur on average, approximately one time every thousand years. The intent of the building code is that structure's experiencing such shaking have a low probability of collapse, on the order of 10% or less. To evaluate the structure's ability to meet this goal, we evaluated the mean demands derived from the suite of seven ground motions, together with those resulting from dead, live and settlement-associated loading with the criteria for collapse prevention performance contained in ASCE 41-06. This criterion is generally accepted as equivalent to the intent of the building code for collapse resistance.

4.3.2 Modeling

The nonlinear model include mathematical representation of the reinforced concrete shear walls, moment frames coupling beams, basement retaining walls, concrete slabs at select levels, and the pile cap foundation. Figure 28 is an isometric view of the model. Figures 29 and 30 respectively show elevations of the shear walls and moment frames in the model. Figure 31 is a plan view of the pile cap model.

In the Perform model we provided nonlinear properties for ductile elements that are anticipated to experience high demands, based on our linear analyses. These include:

- Shear walls (flexure and shear behavior) at levels B1-13 at all respective gridlines.
- Shear wall (shear behavior) at levels 17-22 and 42-46 on gridlines C and F.
- Reinforced concrete columns (flexural hinge) at all levels and respective gridlines.
- Reinforced concrete beams (flexural hinge) at all levels and respective gridlines.
- Embedded steel beams (shear hinge) at all levels and respective gridlines.
- Concrete pile cap modeled as a grillage of beams (flexure and shear hinges).
- Vertical soil springs (axial compression).

We used linear representation of all other elements. The sections below present brief discussion of the modeling assumptions.

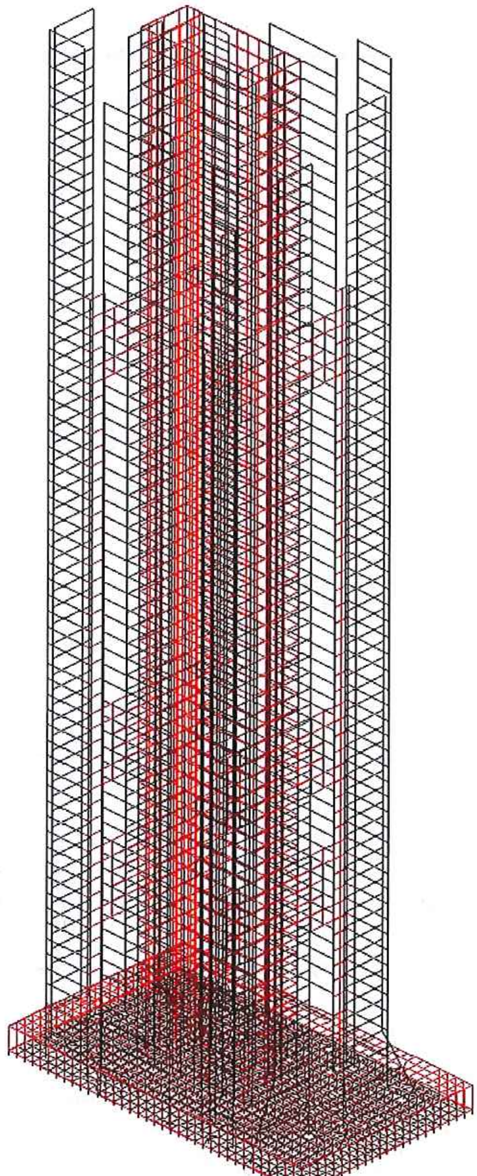
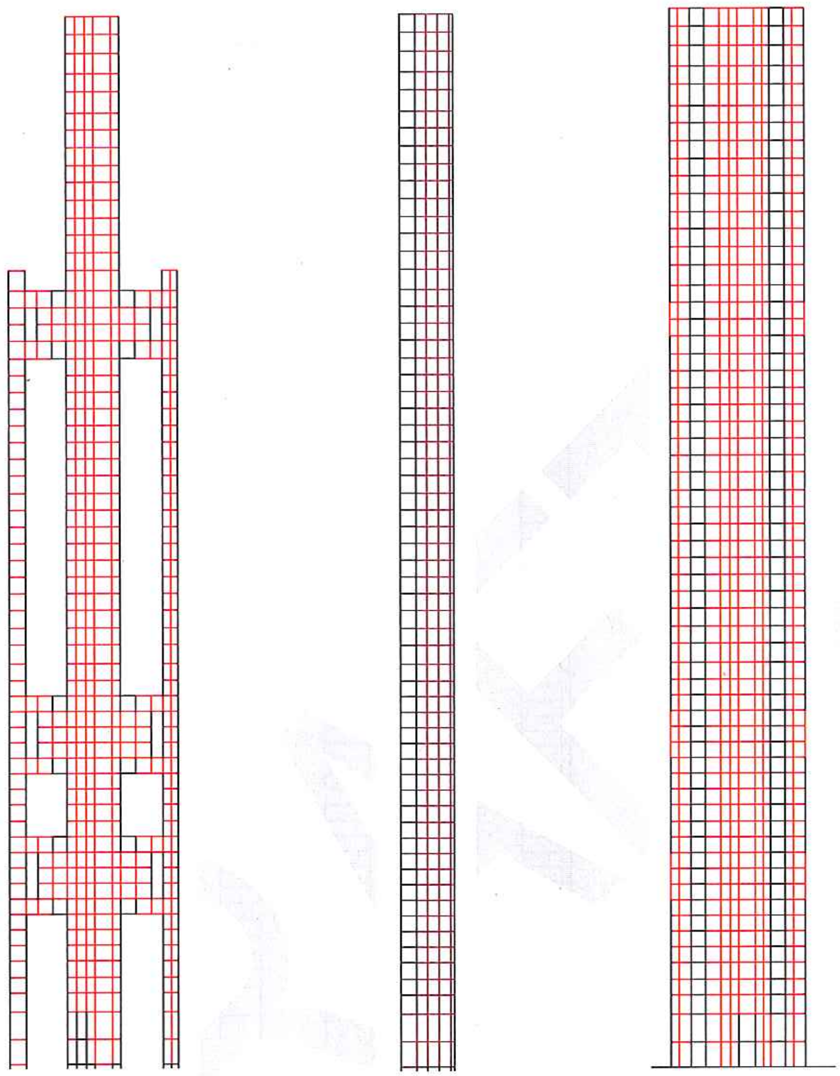


Figure 28 - Isometric view of Perform model



(a) (b) (c)

Figure 29 - Wall elevations (a) Lines C, F; (b) Lines C.7, E.3; and (c) Lines 4, 9.

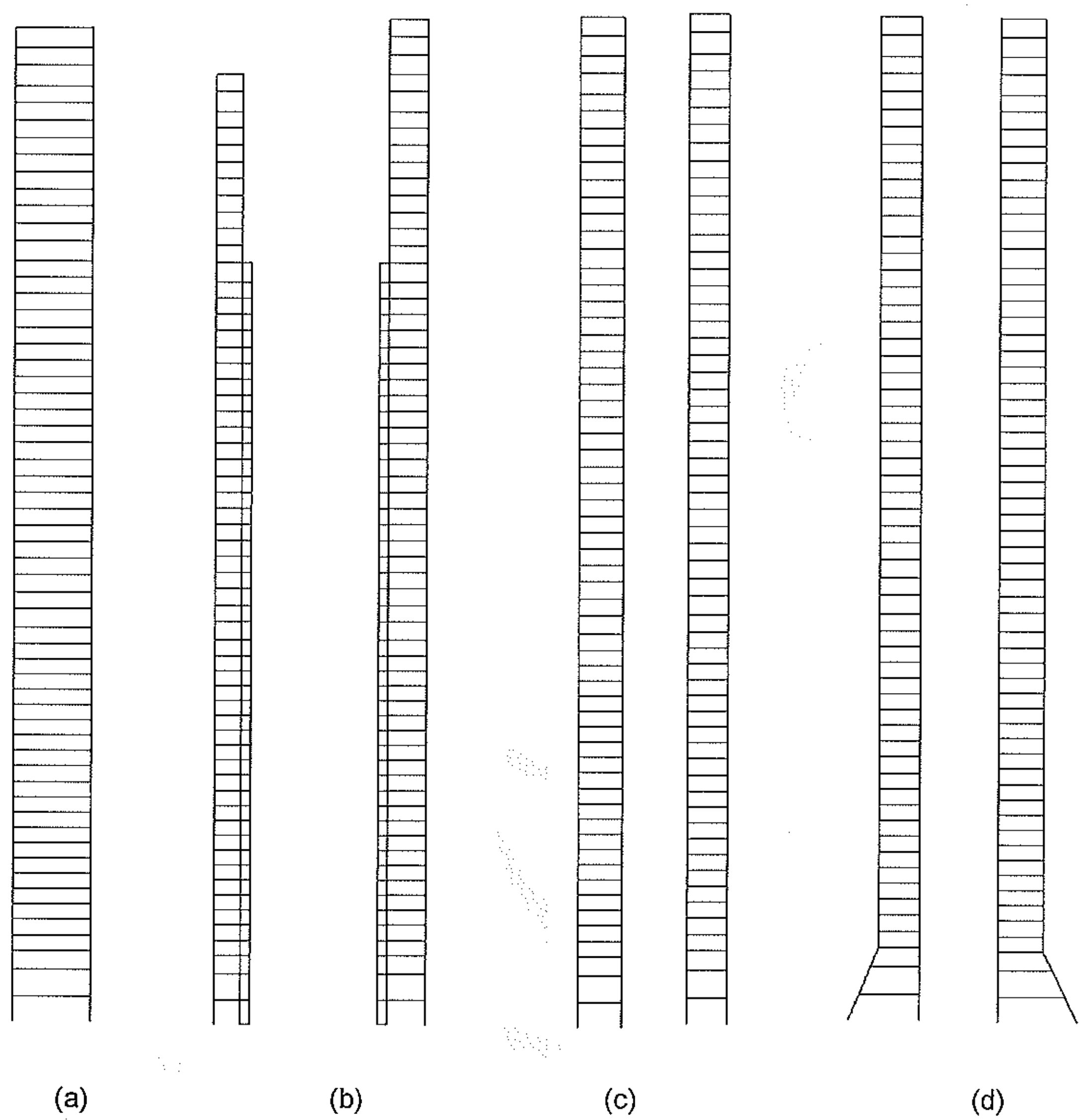


Figure 30 - Frames (a) Lines 1, 12; (b) Lines 2, 11; (c) Lines A, A.2; and (d) Lines G.8, H.

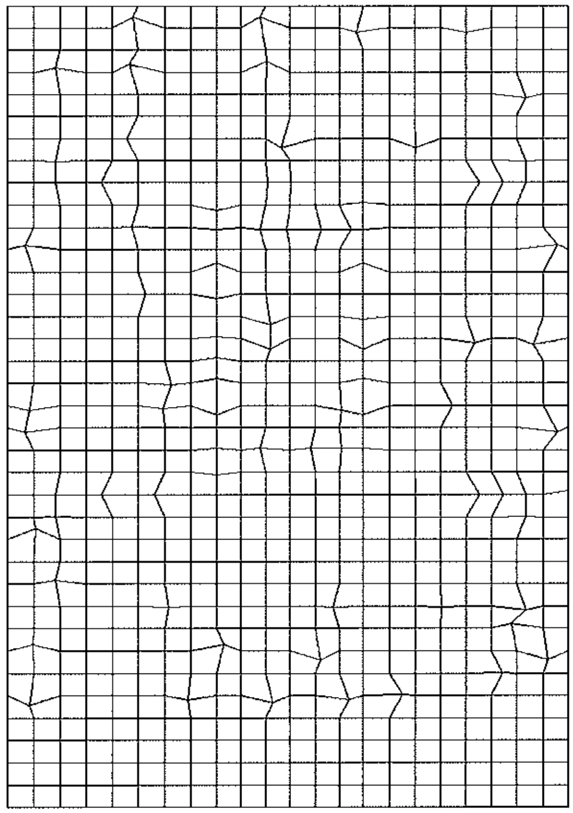


Figure 31 - Plan view of pile cap model

4.3.2.1 Core Shear Walls

We modeled the core shear walls as planar sections of wall discretized vertically using a single element per story throughout the entire building height. The walls are typically discretized horizontally into several elements depending on the wall geometric configuration and discontinuities along the building height, as well as intersecting walls and beams. The aspect ratio of the wall elements (in either horizontal or vertical directions) was generally limited to 1:5. We defined the behavior of shear walls using a compound element which included elastic or inelastic shear material behavior and an elastic or nonlinear fiber model for simulation of wall flexure-axial interaction. We included the self-weight of all Tower walls for mass computation using a concrete density of 150 pcf.

We used nonlinear fiber models for wall segments in the lower thirteen stories. Each fiber model includes eight fibers of reinforcing steel and confined concrete with expected material behavior. We based expected material properties on the Pacific Earthquake Engineering Research Center (PEER) publication: *Guidelines for Performance-based Seismic Design of Tall Buildings*.

We defined the confined concrete material using the Inelastic 1D Concrete Material with a trilinear curve, strength loss, no tension capacity (zero stiffness and strength), and no cyclic degradation. We excluded the unconfined cover portions of the walls in the creation of the tri-linear curve since the cover region is relatively small in comparison to the wall thickness. This has the conservative effect of under-estimating the walls true strength.

We modeled the shear wall longitudinal reinforcing steel using the *Inelastic Steel Material, Non-Buckling* with different tensile and compressive trilinear curves, strength loss, and no cyclic degradation. We assumed that relatively low residual strength would result from compressive buckling reinforcing steel, while tension strength would not degrade significantly until rebar fracture occurs.

For shear walls above Level 13, which were modeled linearly, we defined elastic axial-flexural behavior using effective cracked cross section properties for in-plane and out-of-plane bending defined as $EI_{eff}=0.5EI_{gross}$.

We monitored compressive and tensile strains at each wall end (defined using either elastic or nonlinear behaviors) using axial strain gages elements. The strains limits for the concrete and reinforcing steel material are defined per section 6.3.3.1 of ASCE 41 as follows:

- Concrete compressive strain limit: 0.005
- Steel compressive strain limit: 0.02
- Steel tensile strain limit: 0.05

We also monitored shear wall rotation using 4-node rotation gages at corner nodes with limit states defined per Table 6-18 of ASCE 41, assuming high axial and shear demands, as well as confined boundary elements.

We defined a nonlinear shear model for shear walls and wall segments in the lower thirteen stories of the Tower at all wall lines (i.e., gridlines C, C.7, F, E.3, 4, 9, 2, and 11), as well as at the remaining outrigger levels (i.e., levels 17-22 and 42-46 at gridlines C and F).

We also modeled Column B located at gridlines C and F using two intersecting shear wall elements forming an L-shape configuration, as shown in Figure 32. For Column B we used a nonlinear shear model at levels 1-13, 17-22, and 42-46, corresponding to the lower levels, predicted to have high demands in our linear analyses and the outrigger levels. For all remaining wall locations (including Column B) we defined an elastic shear material behavior with an effective elastic stiffness, $GA_{v,eff}=0.5GA_{v,gross}$.

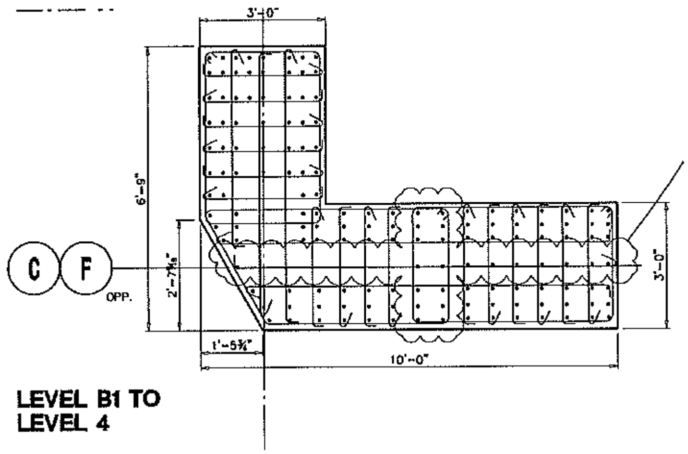


Figure 32 - Column B cross section modeled as L-shaped shear wall

Since the Tower core shear walls also act as bearing walls with computed axial load demand (including gravity and seismic loads) exceeding $0.15P_o$, where P_o is the wall compressive capacity, shear behavior should be estimated per ASCE 41 as a force-controlled action using nominal (lower-bound) material properties. However, since the walls are detailed in structural drawings for ductile behavior with confined boundary elements and cross ties bracing every vertical bar along the entire wall length, we defined shear wall shear behavior as a deformation-controlled action using a nonlinear backbone curve per Table 6-19 of ASCE 41. For high axial loads the resulting drift and shear stress ratios shown in Figure 33 are g=0.40%, d=0.75%, e=1.0%, c=0.0 (we used 0.05 for numerical stability of the Perform model), and f=0.60. The initial shear stiffness in the nonlinear shear model is also set to 0.5 times the gross stiffness in anticipation of some cracking in the walls.

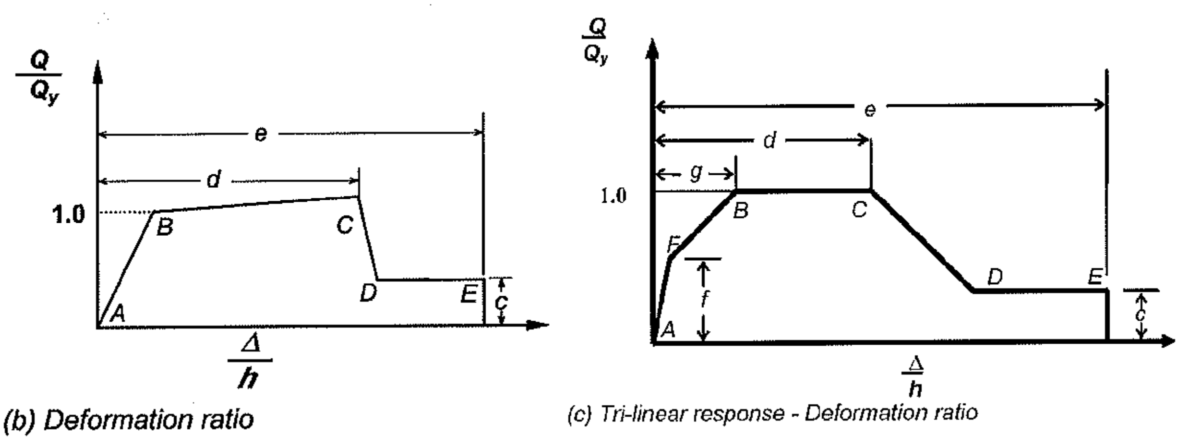


Figure 33 - ASCE 41 generalized shear backbone curve

We used the Perform *Wall Inelastic Shear Material* to define the shear backbone curve using a trilinear relationship, strength loss, and no cyclic degradation. We monitored the shear performance of all shear walls using 4-node shear gauges at corner nodes and compared these to the ASCE 41-06 Collapse Prevention limit state for secondary components (since strength-degradation is modeled throughout the Tower) defined in accordance with Table 6-19 as a limiting drift ratio of 1.0%.

4.3.2.2 Basement Retaining Walls

We modeled the basement retaining walls at level B1 located at the Northern, Southern, and Western edges of the foundation pile cap using an elastic fiber model and elastic shear material. We defined the effective cracked cross section properties similar to the core shear walls using a 0.5 multiplier for the Elastic Modulus and Shear Modulus.

We monitored the performance of the basement walls similarly to the core shear walls using 4-node shear gauges at corner nodes. We used acceptance criteria consisting of the Collapse Prevention limit state for secondary components (since strength-degradation is modeled throughout the Tower) defined in accordance with Table 6-19 as 1.0% drift.

4.3.2.3 Outrigger Coupling Beams

We modeled the outrigger coupling beams on gridlines C and F as shear wall elements with inelastic axial/flexural behavior and shear behaviors. We used the same modeling approach for the axial/flexural fiber model of the outrigger coupling beams as the core shear walls. We similarly used the Perform *Wall Inelastic Shear Material* to model the shear backbone curve using a trilinear relationship, strength loss, and no cyclic degradation. However, we modified the shear stress, strain parameters and limit states for these coupling beams to account for the diagonal reinforcement present.

Initial shear stiffness of the coupling beams is defined as a cracked effective stiffness of $GA_{v,eff}=0.5GA_{v,gross}$, modified by applying a 0.5 multiplier to the Shear Modulus of the concrete material at the corresponding level. We estimated the ultimate shear strength $V_{n,Total}$ as the added strength due to diagonal reinforcement, V_{nd}, shear wall vertical reinforcement, V_s, and concrete contribution, V_c, not to exceed a limiting shear strength for shear walls of $10\sqrt{f'_c}A_{cv}$. Per Table 6-19 of ASCE 41 for walls with high shear demands, strength loss in conventionally-reinforced wall coupling beams with conforming transverse reinforcement is expected at a shear strain of 1.6%. Therefore, we assumed that beyond this drift limit the remaining strength in the coupling beams would only be attributed to the diagonal reinforcement, V_{nd}. Note that the shear strain and deformation demand in the coupling beams occurs in the vertical direction. Since diagonal reinforcement provides a highly ductile behavior, we also assumed that failure of the coupling beams at the Collapse Prevention levels would only occur at shear strain (drift) demands of approximately 6.0%.

4.3.2.4 Reinforced Concrete Columns

We modeled the reinforced concrete columns in moment frames on gridlines 1, 2, 11, 12, A, A.2, G.8, and H using the *FEMA Columns, Concrete Type* with symmetric elastic-perfectly-plastic behavior, strength loss, deformation capacities, and no cyclic degradation. We defined flexural plastic hinges at both ends of the columns, assuming an inflection point at mid-span. We used Table 6-8 of ASCE 41 to define the parameters of the nonlinear hinge model assuming ACI-conforming details with 135° hooks, high axial and shear demands as computed in preliminary analysis results, as well as a high transverse reinforcement ratio as shown in structural drawings. For each column type we determined if the behavior is shear-controlled or flexural-controlled by comparing the plastic shear capacity, V_p (i.e., shear demand at flexural yielding of plastic hinges), and the nominal shear capacity based on transverse reinforcement detailing, V_n. We used the program SpColumn v4.81 to determine the flexural capacity and axial load - moment interaction diagram for each column in the weak- and strong-axis directions. We used expected material properties for concrete and steel reinforcement. At lower levels, due to relatively high longitudinal reinforcement ratios, columns were determined to be shear-controlled (condition iii with $V_p/(V_n/k)>1.0$).

We used effective cross section properties to define the elastic behavior of the columns by applying a 0.5 multiplier to the strong-and weak-axis bending inertias of each column, and a 0.7 multiplier to the axial area to account for expected cracking at the bottom 20 stories. Above the 20th-story we applied a 0.6 multiplier to the axial area.

The resulting ASCE 41 parameters (Table 6-8) defining the nonlinear behavior of these column hinges are a=0% rad, b=0.8% rad, c=0% corresponding in Perform to DL, DX, and FR/FU, respectively. We initially monitored the performance of the columns using a limiting plastic hinge rotation of 1.0% for flexural-controlled columns and 0.8% otherwise, a limit state corresponding to the Collapse Prevention level per ASCE 41 for primary elements modeled with strength degradation.

In initial nonlinear analyses we detected high rotational demands at lower levels. Following this observation, we recomputed the actual plastic rotation capacity of select columns using the XTRACT v3.0.7 program. We used expected material properties and transverse reinforcement details to calculate confined concrete stress-strain relationships. Table 2 summarizes the results for select columns.

Table 2: Expected plastic rotation capacity of concrete column based on M-ϕ analysis

Column	Gridlines	Levels	Maximum axial load ratio (P_{max}/f'_cA_g)	Plastic rotation, θ_p (rad)	
				M-ϕ analysis	ASCE 41 criteria
A	1,12	B1-L4	41%	1.81%	0.8%
C	2,11	B1-L4	57%	1.15%	0.8%
D	A, A.2, G.8, H	B1-L3	40%	1.65%	0.8%

4.3.2.5 Reinforced Concrete Beams

We modeled reinforced concrete beams in moment frames on gridlines 1, 2, 11, 12, A, A.2, G.8, and H, as well as the outrigger spandrel beams on gridlines C and F and levels 8, 13, 17, 22, 42, and 46 using the *FEMA Beam, Concrete Type* with symmetric elastic-perfectly-plastic behavior, strength loss, deformation capacities, and no cyclic degradation. We defined flexural plastic hinges at both ends of the beams, assuming an inflection point at mid-span. We used Table 6-7 of ASCE 41 to define the parameters of the nonlinear hinge model conservatively assuming high shear demands, as well as similar negative and positive reinforcement ratios and conforming transverse reinforcement, as shown in structural drawings. The resulting parameters are a=2.0% rad, b=4.0% rad, c=20% corresponding in Perform to DL, DX, and FR/FU, respectively. We monitored the performance of the beams using a limiting plastic hinge rotation of 4.0% corresponding to the Collapse Prevention level per ASCE 41 for primary elements modeled with strength degradation.

We used effective cross section properties to define the elastic behavior of the beams by applying a 0.5 multiplier to the strong- and weak-axis bending inertias of each beam to account for expected cracking.

4.3.2.6 Embedded Steel Coupling Beams

We modeled the embedded steel coupling beams spanning core shear wall segments in the Tower's longitudinal direction on gridlines 4 and 9 as nonlinear shear-controlled beams. We confirmed this behavior through preliminary analysis results. We used the *Shear Hinge, Displacement Type* element in Perform and assigned it to the beam mid-span. Outside the shear hinge, we used a steel beam cross section per structural drawings with elastic material properties and a reduction factor of 0.6 applied to the strong-axis bending inertia. We modeled embedded beams with zero mass and increased flexural stiffness in the model to simulate the continuity of coupling beams at wall supports.

The coupling beam nonlinear shear behavior including element stiffness, yield, and degradation characteristics are matched to coupling beam testing program performed by Dr. John Wallace at UCLA (*Experimental Evaluation and Analytical Modeling of ACI 318-05/08 Reinforced Concrete Coupling Beams Subjected to Reversed Cyclic Loading*, UCLA – SGEL Report 2009/06, August 2009; *Large-Scale Testing and Analysis of Concrete Encased Steel Coupling Beams under High Ductility Demands*, 15th World Conference on Earthquake Engineering, September 2012).

We defined the limiting shear hinge displacement corresponding to the Collapse Prevention level as the shear displacement at initiation of strength loss, DL.

4.3.2.7 Floor Diaphragms

We used rigid diaphragm constraints to model floors above the first level. We represented the floor slab on Level 1 (Ground level) using elastic shell elements acting as a membrane, i.e., with in-plane axial and bending stiffness and zero out-of-plane bending stiffness. We used effective cross section properties for in-plane bending/axial of the slab by applying a 0.35 multiplier to the concrete Elastic Modulus. The first floor slab is connected throughout the floor area to the Tower shear walls and columns above and below level L1, while the slab perimeter nodes are connected to underlying retaining walls at basement level B1.

We assigned lumped translational mass at the center of mass of each diaphragm between Levels L2 and Roof based on the ETABS model output provided by DeSimone. We estimated the translational mass at each diaphragm levels based on the floor dimensions and wall layout, and tried to match the torsional vibrational modes obtained from the Perform model with those obtained in from our ETABS model. For Level L1 we assigned translational masses to the floor nodes, based on their tributary area and connecting elements allowing automatic computation of rotational mass. A summary of modal analysis results obtained from Perform and ETABS models corresponding to cracked elastic behavior of the Tower are presented later in the report.

4.3.2.8 Pile Cap

The Millennium Tower is supported on a single, continuous 10 ft thick cast-in-place reinforced concrete pile cap connecting precast concrete piles spaced at 4ft-8 in. on center. The 10 ft portion of the Tower foundation extends from gridline A-J.1 in the North-South direction and gridlines 1-12 in the East-West directions. On the South, a PG&E vault is supported on a 3 ft thick slab cantilevered off of the pile cap from gridline J.1 and extending to gridline K. This portion of the mat is directly supported on soil.

Since the high utilization ratios determined in our linear analyses, described above, indicated nonlinear behavior is expected in the pile cap, we developed a nonlinear grillage model of the entire foundation system. We used a relatively regular and orthogonal layout of beam elements representing segments of the pile cap at an approximate spacing of 5 ft on center in both longitudinal (North-South) and transverse (East-West) directions of the mat. This spacing corresponds to the spacing of precast piles throughout the mat North of line J.1. As seen in F. 34? Figure 33 above, the regular beam layout was slightly distorted in some locations to match shear wall and column layouts and to provide nodal points at the 31 points at which settlement measurements are available.

The selected grillage beam spacing results in very deep rectangular beams measuring 60 in. width (equal to beam spacing at 5 ft on center) by 120 in. deep (corresponding to a 10 ft thick slab). We evaluated the validity of this modeling approach by comparing the flexural and shear force-deformation relationship of simple elastic beam models in Perform to a thick mat foundation modeled using finite element analysis (FEA) in Abaqus v6.13.1. We assessed two different beam lengths: 60 in. (5 ft beam spacing in orthogonal direction) and 240 in. (4 beams in series arbitrarily selected to represent a longer beam span). In the Abaqus model we used solid 3D deformable elements with 8 integration points and a mesh size of 5x5x5 in., as shown in Figure 34. The elastic material properties (E, ν) matched those used in Perform.

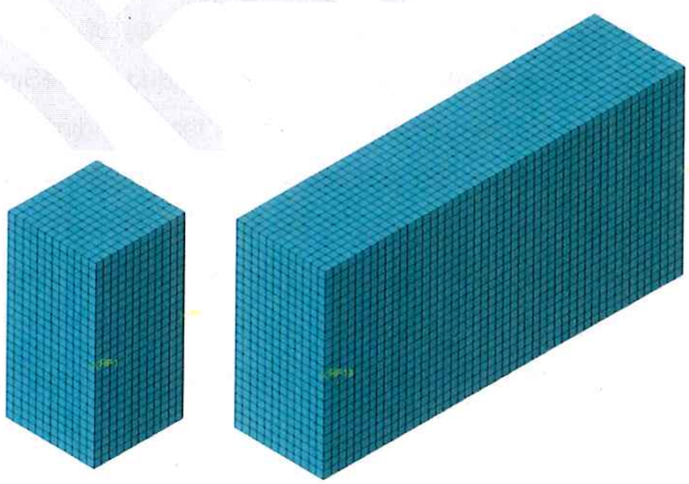
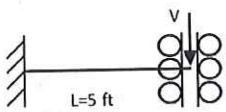
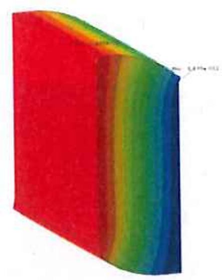
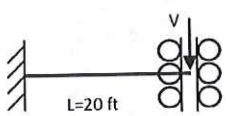
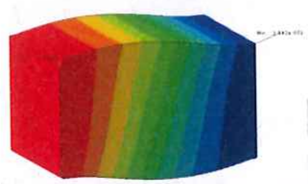
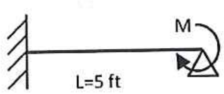
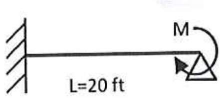


Figure 34 - Rendering of Abaqus models f short and long beam/slab segments

We defined the boundary conditions with one fixed support in all 6 degrees of freedom while the other beam end had an applied shear load, V, or bending moment, M. Under shear load the beam end was allowed to move vertically (with all other degrees of freedom restrained). Under bending moment the beam end was allowed to rotate (with all other degrees of freedom restrained). Table 3 compares the results obtained from the Perform and Abaqus models indicating close agreement and suggesting that the beam elements used in the Perform model are able to adequately capture the mat behavior.

Table 3: Comparison of Perform and Abaqus Results

Model	Deformation Results		
	FEA (Abaqus)	Perform-CSI	Difference
 V=1200k	 $\delta_{vertical}$=1.099e-2 in.	$\delta_{vertical}$=1.18e-2 in.	7%
 V=300k	 $\delta_{vertical}$=2.542e-2 in.	$\delta_{vertical}$=2.60e-2 in.	2%
 M=70,000 kip-in	 θ =1.432e-4 rad	θ =1.477e-4 rad	3%
 M=70,000 kip-in	 θ =2.634e-4 rad	θ =2.673e-4 rad	1%

Since the grillage beam spacing is approximately 5 ft on center and the rectangular cross section is also defined with a width of 5 ft, overlap occurs at every grillage intersection. To assure that the grillage model appropriately captured the mat's stiffness we also prepared a shell model of the mat and compared the elastic stiffness of the grillage model to the shell model. To match the deformed shape throughout the pile cap we had to apply a 0.60 multiplier to the concrete elastic modulus ($EI_{eff}=0.6EI_{gross}$). Figure 35 shows the resulting deformed shape of the elastic grillage model when subjected to the building settlements. To create this deformed shape, we placed stiff vertical springs at the locations of the settlement recordings, then applied vertical loads equal to the recorded settlement times the spring stiffness. We then added shear and flexural hinges as well as flexural hinges at each end of each beam segment.

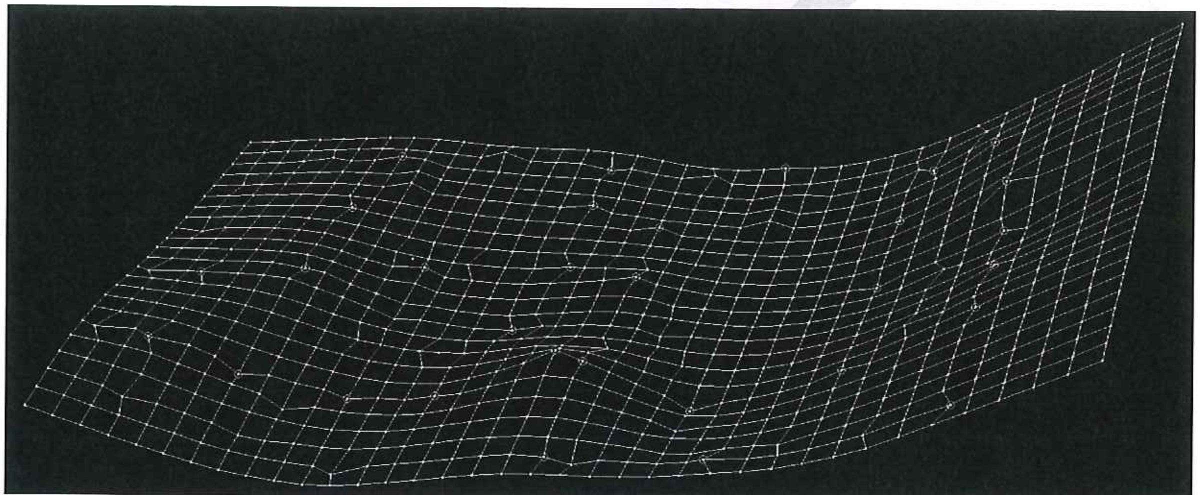


Figure 35 - Exaggerated deformed shape of elastic grillage model with $EI_{eff}=0.6EI_{gross}$.

4.3.2.9 Soil Springs

We added distributed compression-only soil springs throughout the entire pile cap at each nodal point serving as a grid intersection point. These nodal points have a spacing of approximately 5 ft on center, approximately matching the 4 ft-8 in. spacing of the precast concrete piles. We used the Perform Nonlinear Elastic Gap-Hook Bar element for these springs. Initially, we defined a constant spring stiffness throughout the pile cap of 1,800 kip/in corresponding to a high subgrade modulus of 500 pci representing stiff soil conditions and the 5 ft x 5 ft tributary area. As described above, we applied settlement loads as vertical external forces at the locations of available settlement measurements. We increased the stiffness of the beam elements around the point of load application to minimize local deformation associated with these fictitious forces. Figure 36 is an isometric view of the mat with the soil springs present and showing the locations of the applied vertical loads at settlement recording sites. Figure 37 shows the areas of locally stiffened grid elements described above.

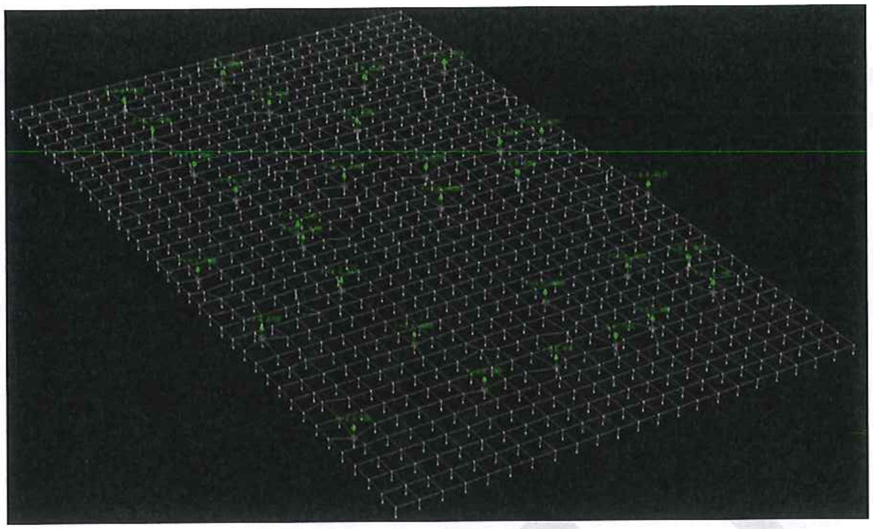


Figure 36 - Pile cap model with soil springs and applied settlement loads

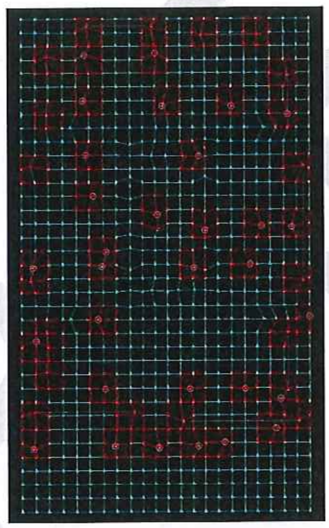


Figure 37 - Locations of stiffened beams in mat model (highlighted in red).

In order to produce the displacement profile shown in Figure 35, we placed very stiff soil springs only at the locations of settlement readings. When we placed distributed springs into the model, the mat redistributed the vertical "settlement" loads across the distributed springs, producing a different settlement profile. To obtain a deformed shape matching the Arup measurements we used an iterative procedure in which we varied both the spring stiffness and applied loads until we obtained an appropriate deformed shape. In addition to the vertical "settlement" loads on the mat, we also applied dead and expected live loads, taken as 25% of the design live load throughout the tower. Figure 38 indicates the final deformed shape we obtained following the above process.

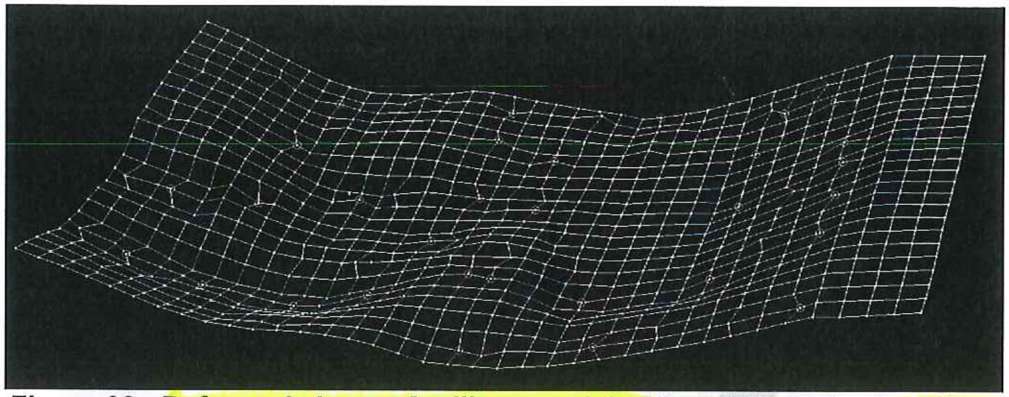


Figure 38 - Deformed shape of grillage model with distributed soil springs

The final soil spring values ranged from 170-1,550 kip/in, corresponding to subgrade modulus values ranging from 50-430 pci. Figure 39 shows the distribution of soil subgrade modulus used to calculate the final soil springs in our model. The resulting mat displacement pattern matched the scaled settlement contours developed by Arup within 10% accuracy.

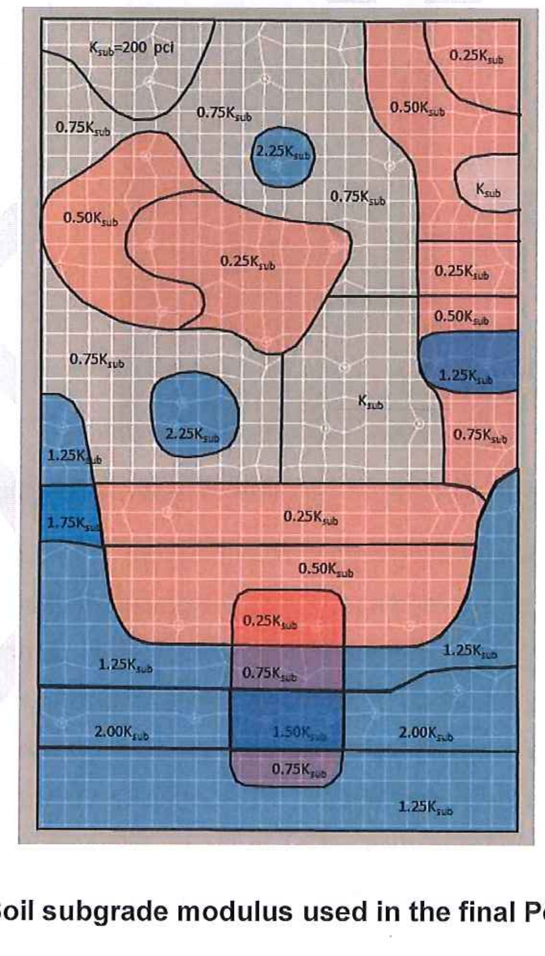


Figure 39 - Soil subgrade modulus used in the final Perform model

4.3.3 Loading

4.3.3.1 Applied Settlement

As discussed in the prior section, we imposed a deformation profile on the pile cap foundation using the differential settlement measurements reported by Arup in their 28 February 2014 report, but scaled to reflect the fact that a portion of the differential settlement occurred during the tower's construction and before the structure gained its full stiffness. Since settlement measurements recorded during construction were not obtained in the same density or at the same points as those taken by Arup during the post-construction period, it was not possible to precisely determine how much settlement occurred at various locations of the mat during construction. Monitoring of the pile cap settlement between January 2007 and February 2009 shows a total settlement of 8.3 in. Therefore, we scaled the measurements obtained by Arup by a 2/3 factor to represent the post-construction portion of the settlement, likely to have created locked-in stresses.

4.3.3.2 Ground Motions

To simulate earthquake demands on the structure, we selected and scaled a suite of seven ground motions to represent Maximum Considered Earthquake (MCE_R) shaking as defined in the present San Francisco Building Code. Although the project geotechnical report prepared by Treadwell & Rollo for the building's original design provide site-specific response spectra, these spectra represented design shaking as defined in the 2001 SFBC. Therefore, we initiated this process by computing an MCE_R acceleration response spectrum for the site using an on-line applet hosted by the United States Geologic Survey and using seismic hazard data compatible with the present SFBC. This applet requires input of site coordinates and site class. We used coordinates of 37°47'25.43" N, 122°23'46.25" W and assumed Site Class D conditions. Table 4 summarizes the spectral parameters we obtained from this applet.

Table 4: Spectral Acceleration Parameters

Parameter	Value
S_s	1.50 g
S_1	0.60 g
$S_{MS}=F_aS_s$	1.50 g
$S_{M1}=F_vS_1$	0.90 g

The Treadwell & Rollo geotechnical report indicates that at long periods, the site-specific spectrum the developed for design of the tower is governed by a building coded requirement that site specific spectra not be taken less than 80% of the standard spectrum defined by the building code. Therefore, we reduced the response spectrum obtained using the USGS spectral parameters using a factor of 0.8 in the long period range. We used the resulting scaled spectrum as a target for scaling the suite of seven ground motions. We applied a least squares approach to derive scale factors that minimized the error between the average spectrum derived from the seven scaled motions and the target spectrum over a period range of 1 second to 7.5 seconds ($0.2T_1$ to $1.5T_1$). Table 5 lists the 7 motions we selected and the scale factors we applied to each. Figure 40 plots the scaled square root sum of square spectrum (SRSS) for each of these seven motions, as well as the average SRSS spectrum. As can be seen, the average SRSS spectrum envelopes the unreduced MCE_R spectrum over the period range 3 seconds to 8 seconds and does not fall below 80% of the MCE_R spectrum in the period range 1 second to 8 seconds.

Table 5: Ground motions and scale factors

GM	NGA#	Scale Factor	Event	Year	Station	M_w	Mechanism	Distance R_{rup} (km)
1	183	2.05	Imperial Valley	1979	El Centro Array #8	6.53	Strike-Slip	3.9
2	143	1.02	Tabas, Iran	1978	Tabas	7.35	Reverse	2.0
3	900	2.74	Landers	1992	Yermo Fire Station	7.28	Strike-Slip	23.6
4	838	4.00	Landers	1992	Barstow	7.28	Strike-Slip	34.9
5	1176	1.33	Kocaeli, Turkey	1999	Yarimca	7.51	Strike-Slip	4.8
6	1530	2.35	Chi-Chi, Taiwan	1999	TCU103	7.62	Reverse-Oblique	6.1
7	1148	4.00	Kocaeli, Turkey	1999	Arcelik	7.51	Strike-Slip	13.5

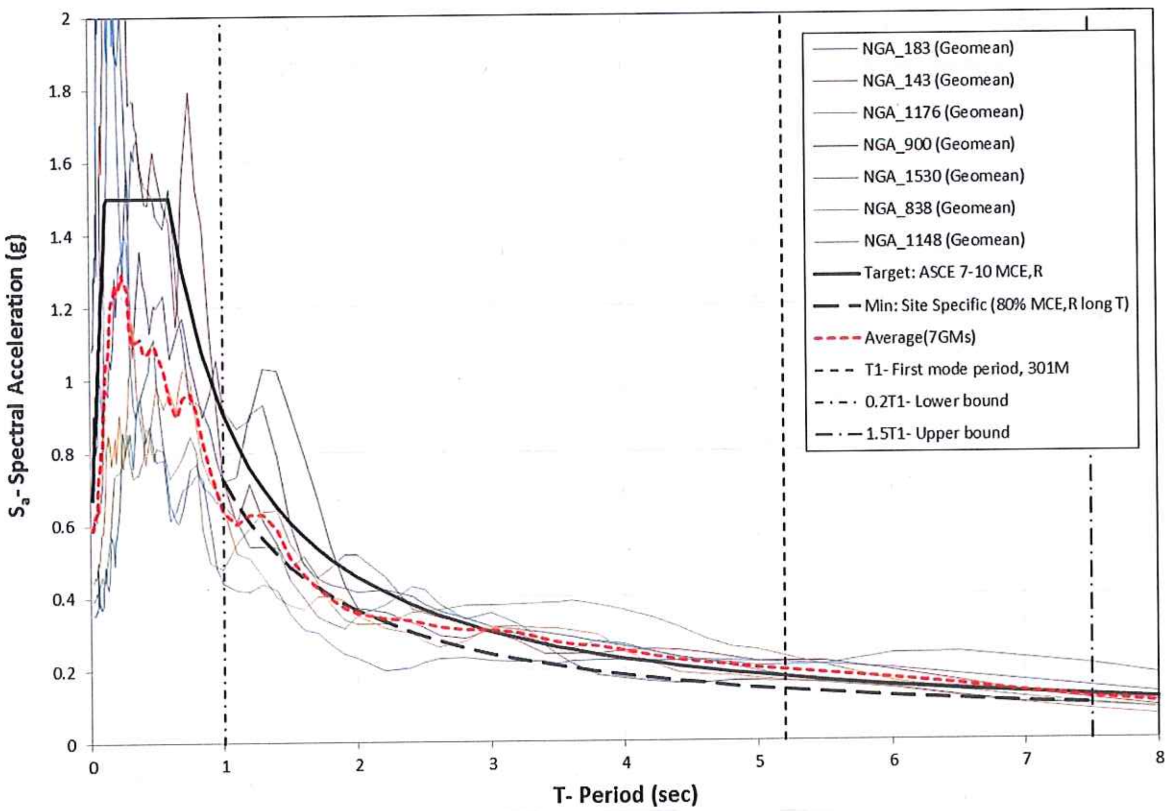


Figure 40 - Scaled ground motions and the MCE_R spectrum

4.3.3.3 Load Combinations

In order to allow evaluation of the effect of foundation settlements on the building's seismic resistance we considered two separate load cases: 1) gravity loads plus seismic response, and, 2) gravity loads, seismic response and settlement. In both load combinations, we combined seismic and gravity loading as recommended by the PEER *Guidelines for Performance-Based Design of Tall Buildings*. The resulting load combinations are

- **No settlement load case**: 1.0D+0.25L+1.0E
- **Settlement load case:** 1.0D+0.25+0.67Set+1.0E

Where D - Dead load

L - Unreduced live load

Set - Measured differential settlements

E - Demand produced by response to each earthquake record

In our Perform models we used the dead load and unreduced live loads defined in the ETABS model developed by DeSimone Consulting Engineers. We used a combination of nodal loads on shear walls, distributed loads on beams, and uniform loads on the first floor slab. We were able to match within 2% accuracy the gravity loads obtained from the two programs. Accidental torsion was not considered.

For the load combination without settlement, we first applied dead and live loads to the model in accordance with the load combination described above. We then subjected the model to each of the seven ground motion records described above and performed nonlinear response history analysis. We used constant modal damping of 3% and negligible Rayleigh damping. We then computed the mean (average) peak demands on the elements of interest from the results of the seven analyses. For the load combination with settlement we followed the same procedure, except that prior to performing the nonlinear response history analyses, we first applied the settlement loads, as previously described.

4.3.4 RESULTS

As with our linear analyses, we evaluated overall building lateral displacements (drifts), axial and flexural demands on beams and columns forming moment frames, shear stresses on shear walls and outriggers and shear and flexural stresses on the pile cap. We discuss each of these below.

4.3.4.1 Lateral Drift

Figure 41 shows the tower's average lateral displacement response from the suite of seven MCE_R ground motions in each of the east-west (H-1) and north-south (H2) directions. Lateral displacement is shown as story drift ratio, which is the amount of lateral displacement in each story divided by the story height. For design of new buildings, an average story drift value of 3% is generally considered acceptable. The 301 Mission Tower displays a maximum story drift ratio of 0.62% without settlement and 0.64% with settlement. Thus, the settlements which have occurred have negligible effect on total building drift in response to MCE_R shaking and the projected drift for the building either with or without the settlement is substantially less than that permitted in the design of new buildings.

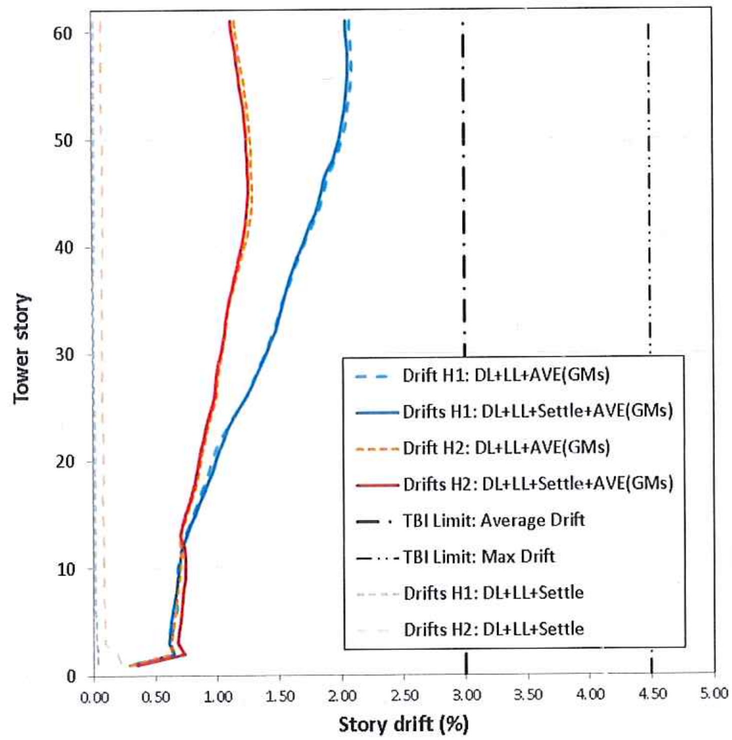


Figure 41 - Story drift - with height

4.3.4.2 Moment Frames

We evaluated the imposed plastic rotation demands on beams and columns forming the reinforced concrete moment frames. Figure 42 is a plot of the maximum plastic rotation demand on any beam in each story of the structure, averaged over the seven ground motions, both with (solid line) and without (broken line) settlement. For design of new buildings, a plastic rotation demand of 4% is considered acceptable. The maximum computed plastic rotation demand for any beam at any level with settlement is 0.14% which is an order of magnitude lower than permitted for design of new structures.

We also evaluated plastic rotation demands on concrete columns forming the moment frames. Depending on the axial load present on the column permissible plastic rotation demand varies from 0.8%, for columns with high axial load to 1.8% for columns with low axial load. Generally, columns throughout the structure have no plastic rotation demand, except at the ground floor level. Without settlement, maximum column base rotation demands are 62% of their capacity. Because foundation settlement has resulted in some rotation of the plane of the top of the pile cap, this rotation is imposed on the columns as well and increases rotation demands on these elements. Considering the combined effect of earthquake loading and settlement, the most severely loaded column has demands as high as 85% of its capacity.

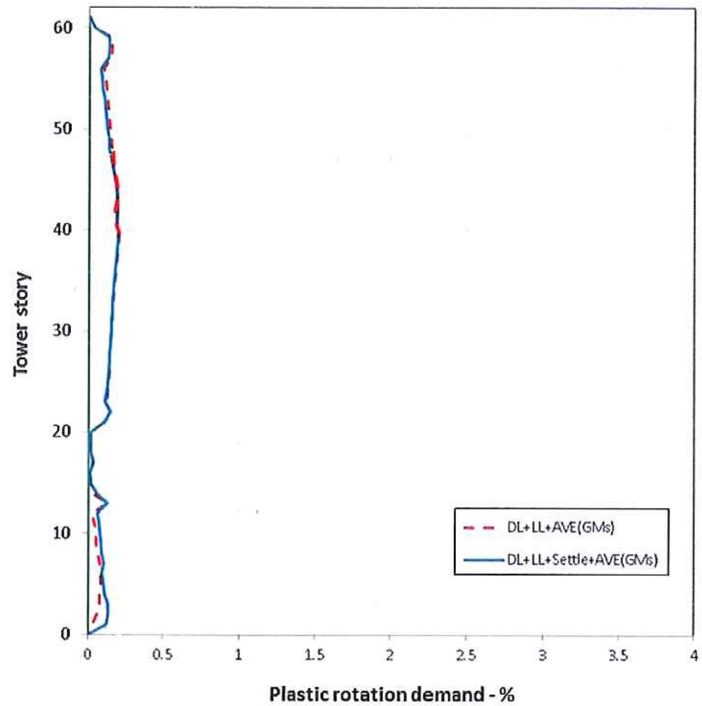


Figure 42 - Moment frame beam plastic rotation demand

4.3.4.3 Shear Walls and Outriggers

Shear walls provide the building's seismic primary seismic force resistance for the building. We evaluated concrete compressive strain, reinforcing steel tensile strain and wall shear strain both with and without settlement. Table 6 summarizes the utilization ratios for each of these quantities. A utilization ratio of 1.0 is considered acceptable.

Table 6: Utilization Ratio for Shear Walls

Response Parameter	Limit	Utilization Ratios No. Settlement	Settlement	Increase
Concrete Compressive Strain	0.005	0.40	056	40%
Reinforcing tensile Strain	0.05	0.04	0.09	125%
Shear Strain	1.0%	1.17	1.27	9%

Settlement increases concrete compressive strain and reinforcing tensile strain significantly, however, both quantities remain substantially below values considered acceptable for new buildings. Settlement increases the maximum shear strain demand on the shear walls by slightly less than 10%. Shear strain demands on the walls is modestly higher than is typically considered acceptable for new buildings before settlement. The small increase in demand in this quantity does not change this.

A series of concrete-encased structural steel beams extend over the openings in shear walls at entryways to the elevator lobbies. Without settlement, shear strain demands on these coupling beams are 79% of their capacity. With settlement, these demands increase slightly to 81% of capacity, but are still within acceptable values.

At the 8th through 12th, 17th through 21st and 42nd through 45th levels, the shear walls extend outward from the core to columns at the building perimeter to serve as outriggers, as shown in Figure 29(a), above. These outriggers are penetrated by a series of openings, to allow occupant passage through the wall lines. Reinforced concrete beams, herein termed outrigger beams, span over these openings. Our analyses indicate that without settlement, the most severely loaded of these outrigger beams experiences demands that are 40% higher than their capacity. Settlement actually reduces these demands by a small amount, modestly improving the building's potential earthquake performance.

4.3.4.4 Pile Cap

We evaluated the pile cap under gravity loads plus settlement alone, under gravity loads plus earthquake response and under gravity loads plus settlement and earthquake response. We determined that under the influence of gravity loads and settlement, the pile cap has likely experienced some inelastic demands, resulting in flexural cracking of the concrete and localized yielding of the reinforcing steel. All of these inelastic demands are within acceptable values and to not jeopardize the pile cap's load carrying capacity. Figure 43 is a plot of the pile cap model showing the locations (shaded in green) of these inelastic demands. These primarily occur at the north and west sides of the central core. All of these demands are in flexure rather than shear.

Figure 44 presents a similar plot of the pile cap model showing utilization ratios under gravity loads plus earthquake but without settlement. Figure 45 is a similar plot, for the load case including gravity load, earthquake and settlement. In these two figures, element highlighted in green are anticipated to experience flexural yielding, elements highlighted in blue are expected to experience flexural yielding in an amount up to twice the value generally considered acceptable for the design of new buildings and elements highlighted in red are anticipated to experience yielding in an amount in excess of twice the amount generally considered acceptable for design of new buildings.

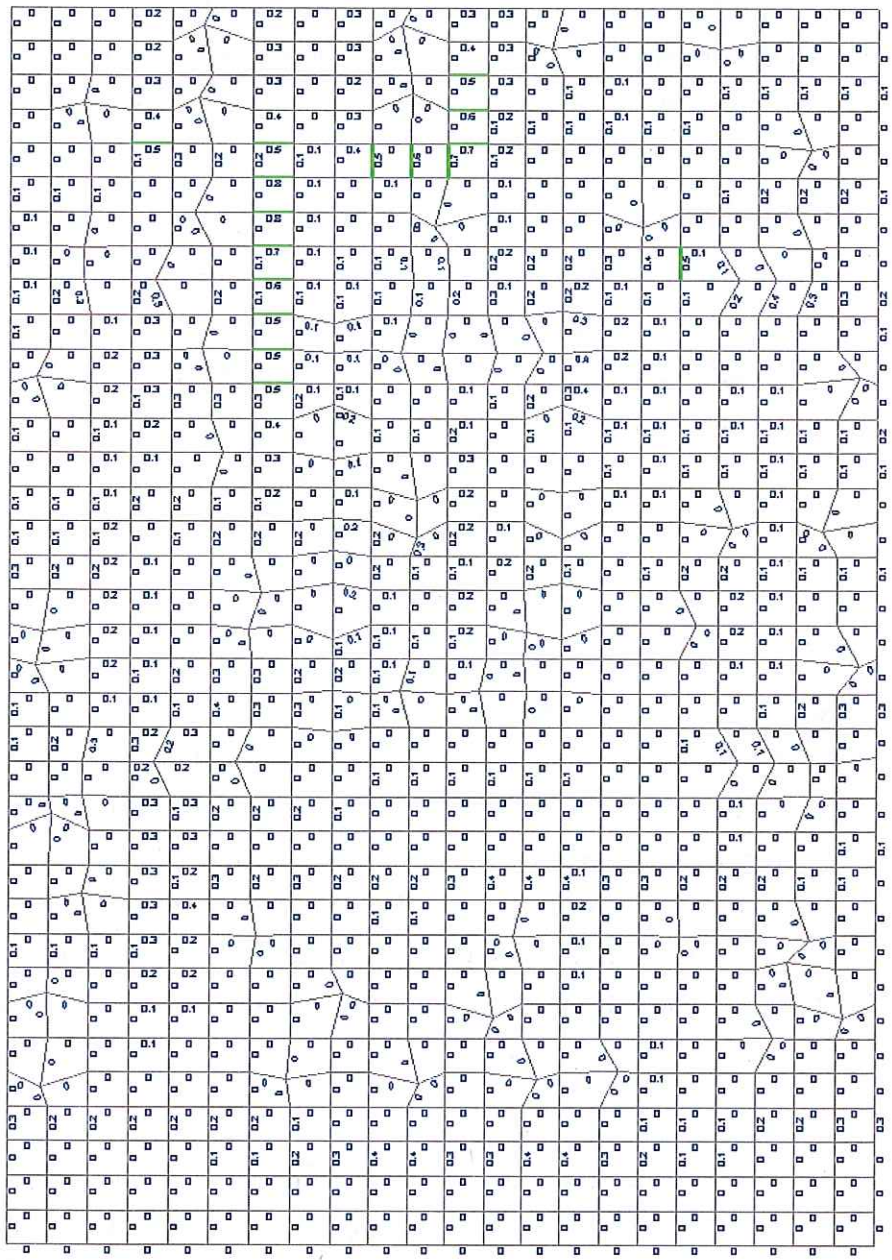


Figure 43 - Pile cap utilization ratios under gravity loads and settlement

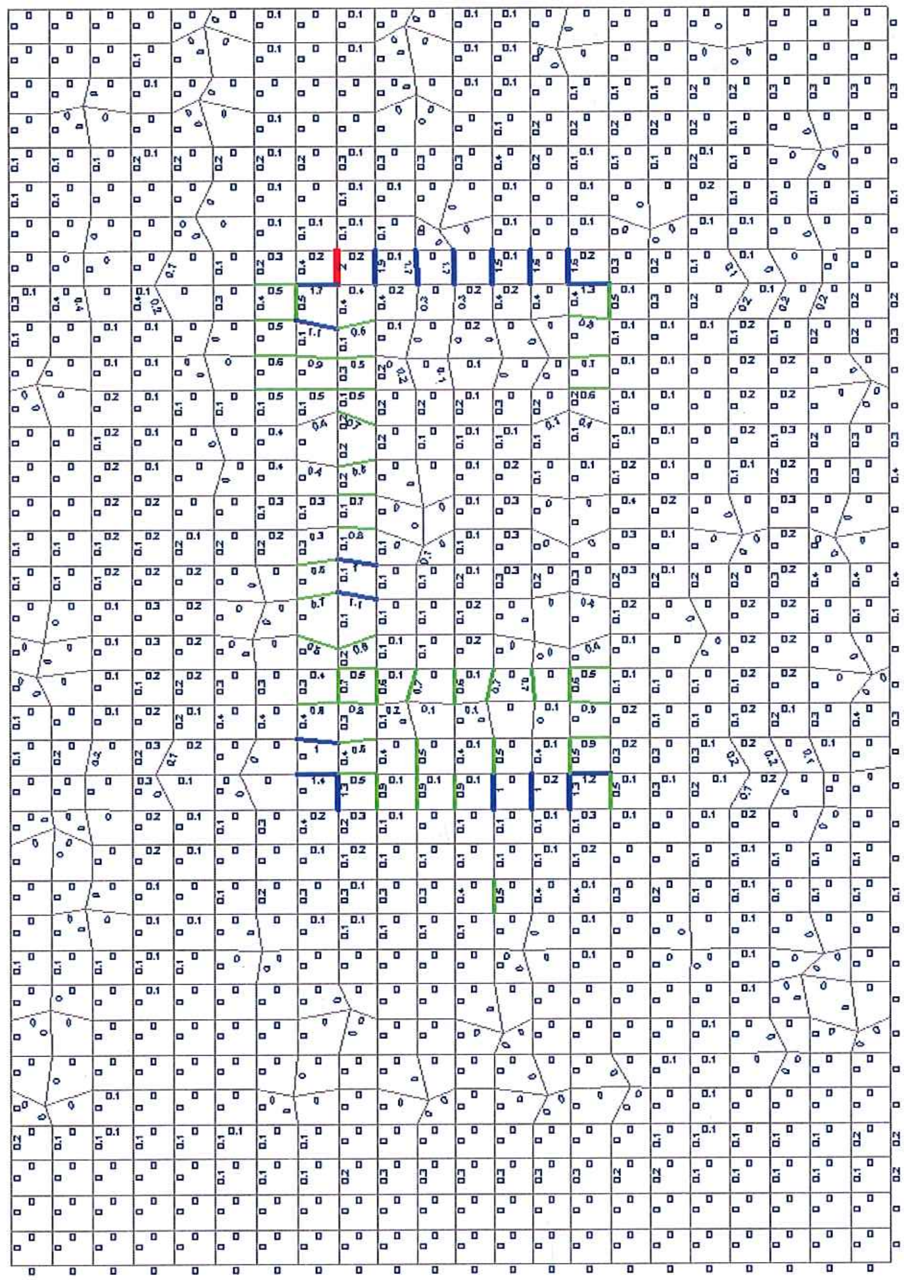


Figure 44 - Pile cap utilization ratios under gravity load and earthquake

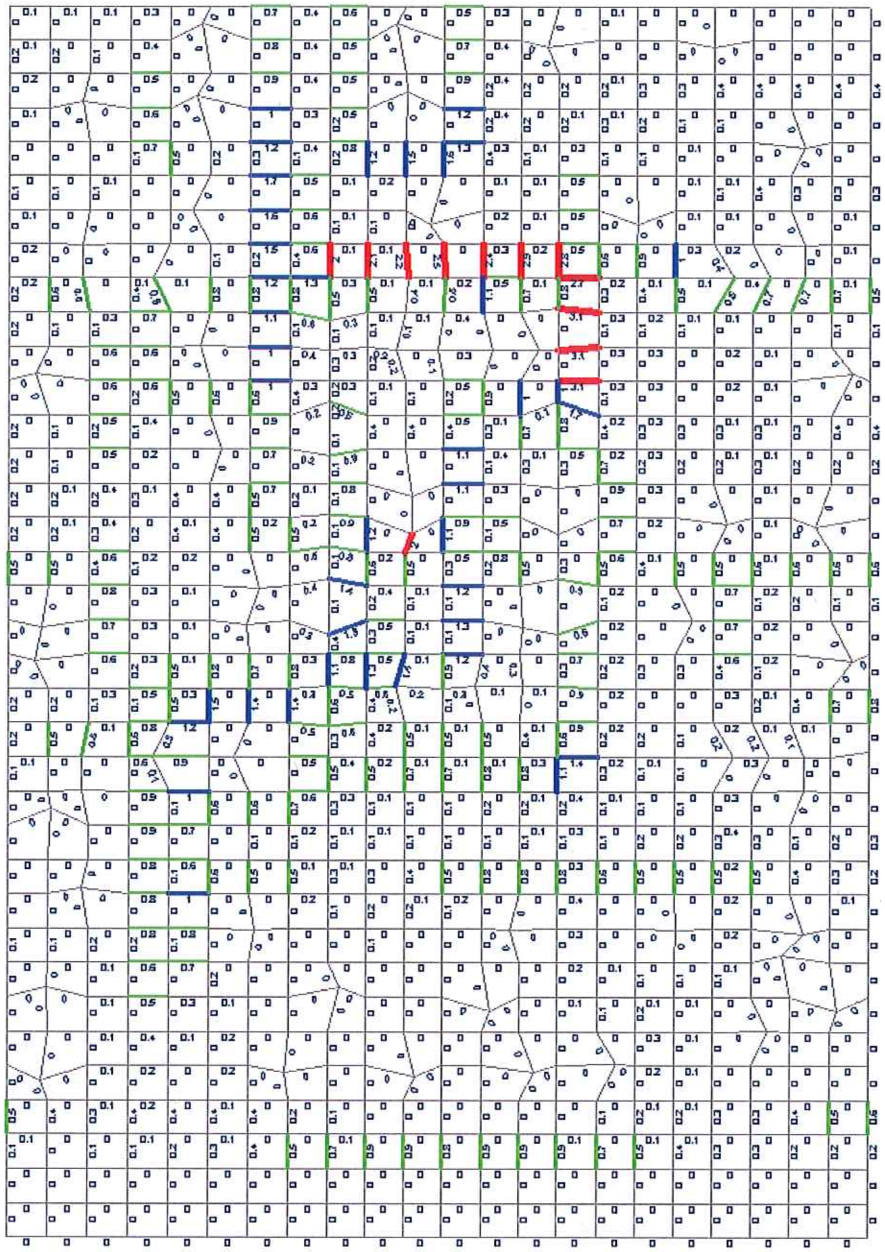


Figure 45 - Pile cap utilization ratios under gravity load, settlement and earthquake

As shown in the figures, for the case of MCE_R earthquake demands without settlement, the pile caps experiences general flexural yielding of the mat around the perimeter of the shear core. Anticipated flexural demands are within acceptable levels along the east and west faces of the core, but exceed levels generally considered acceptable for design at the north and south faces of the core. The locations of excessive yielding are generally limited to the same locations as the case without settlement, however, the demands are somewhat more severe when the presence of settlement is considered.

Under the influence of settlements that have occurred, the pattern of flexural yielding extends throughout the field of the mat.

5. DISCUSSION

Our linear analysis of the effect of settlement on the 301 Mission Building indicates that most structural elements do not experience appreciable stress as a result of these settlements. This includes the load bearing walls surrounding the building's core, which provide most of the building's resistance to earthquake loads, as well as the floor slabs, most columns and beams. The linear analysis does indicate, however, that a few building elements including some columns, some beams in moment frames, outrigger beams, and the foundation itself do experience significant stress as a result of the settlement that has occurred. With the exception of the foundation, none of the settlement-related demands on these elements are at levels that indicate impending failure. Computed stresses at localized areas of the foundation mat, primarily at the central shear walls and in the vicinity of the outrigger frames are quite high and suggest that some cracking of the concrete mat and yielding of reinforcing steel may have occurred.

Our visual observations of the building confirm that most structural elements exhibit no indications of structural distress. Exceptions to this are several beams and walls at the base of the building which exhibit cracking. These beams are located in areas predicted to have high stress in our analyses. Areas of the foundation mat that we observed did not exhibit any signs of distress.

Although our linear analysis and visual observations indicate that the effects of building settlement are quite moderate, linear analyses are incapable of accurately predicting seismic performance. Therefore, we performed nonlinear analysis of the structure to compare the effect of the settlements on the structure's seismic resistance.

On the basis of our nonlinear analyses, we confirmed that the effect of settlement on the building's ability to resist large earthquakes is small. Specifically, we evaluated the building for the combined effects of dead and live loads, settlement and the effects of Maximum Considered earthquake shaking. Though not the worst shaking that could ever occur, Maximum Considered Earthquake shaking is the most severe shaking considered by the building code for design of new buildings. At the 301 Mission site this level of shaking is anticipated to occur approximately 1 time every 1,000 years. The building code anticipates that code-conforming structures subjected to such shaking will experience substantial damage and may not be repairable. The building code also anticipates as much as a 10% chance that code-conforming structures would collapse in such an event.

We compared the demands (drifts, stresses, deformations) on various structural elements against criteria commonly used today to design tall buildings to meet the building code's performance expectations. We determined that most of the building's elements are capable of meeting these criteria. A limited number of elements do not meet these criteria, however, they would not meet these criteria even if no settlement had occurred and the effect of settlement on these elements is very small. Specifically, shear strain demands on selected shear walls are increased approximately 1% by the settlement; shear strain demands in outrigger coupling beams are increased 9% by the settlement while plastic rotation demands on outrigger spandrel beams are slightly decreased by the settlements. These effects of settlement are not significant to the building's performance.

Settlements do significantly increase plastic rotation demands at some locations in the foundation mat, by as much as 50%. It is worth noting that the original design of the mat anticipated the formation of these so-called plastic hinges with the intent that these hinges would act a type of fuse to shield the rest of the structure from damage. The amount of hinging that would occur, however, was never quantified during the original design. Our analyses suggest that the amount of such hinging is more than would generally be considered desirable for a new building. However, because hinging does not fully form such hinges at all possible locations, we do not believe that this represents a collapse hazard.

It may be feasible to locally reinforce the foundation mat to reduce the amount of hinging to a level comparable to that which our analyses suggest would occur in the absence of settlement. Such reinforcement could potentially be accomplished by introducing new walls, at selected locations between the basement and 1st level. Generally, these new walls would be located around the building core and would extend outward, for a length of approximately 20 ft from the core towards the building perimeter.

6. CONCLUSIONS

We conclude that the settlements experienced by the 301 mission tower have not compromised the building's ability to resist strong earthquake shaking within the performance expectations inherent in the building code. However, some limited number of elements, including the building foundation, has experienced significant stresses as a result of the settlement. These elements will experience damage in somewhat lower intensity earthquakes than would be anticipated if no settlement had occurred. We do not believe the safety of building occupants is significantly affected.

I:\SF\Projects\2014\147041.00-301M\WP\001ROHamburger-R-147041.00.jdi_Report_Draft.docx