301 Mission Street - Voluntary Foundation Retrofit
Engineering Design Review Team (EDRT) - Log

Review Panel Members

GD	Greg Deierlein, Chair
MS	Marko Schotanus, R&C
CS	Craig Shields
SV	Shah Vahdani

Construction Documents Phase		Issue and Revision Dates
1	Calcs V1 - Design Overview	12/3/2018, through revision 6 (8/21/2019)
2	Calcs V2 -Gravity	12/3/2018; through revision 5 (6/7/2019)
3	Calcs V3 - Lateral	12/3/2018; through revision 5 (6/7/2019)
4	Calcs V4 - Details	12/3/2018; rhrough revision 4 (5/20/2019)
5	2018_11_30_Geotech Report Combined	11/30/18
6	SGH-301Mission_Permit_Shoring 12-05-2018	12/5/18
7	SGH - 301 Mission_Permit 12-03-2018	12/3/18
8	SGH - 301 Mission_Permit Test Program	12/5/18
9	301 Mission Street-Perimeter Pile Civil-2018.11.29	11/29/18
10	2018-12-05 Shoring Design Report Permit Submittal	12/5/18
11	ROHamburger _ Basis of Design (Oct. 9 2018)	10/9/18
12	SGH-301 Mission Specifications 2018-12-03	12/3/18
13	SGH Responses 3_12_2019 Perimeter Pile Upgrade EI	3/12/19
14	SGH Responses 4_19_2019 Perimeter Pile Upgrade EI	4/19/19
15	SGH Responses 6_7_2019 Perimeter Pile Upgrade ED	6/7/19
16	Slate Settlement Memo (5/21/2019)	
17	Slate draft table of material properties (5/22/1019)	
19	Slate - lateral earth pressures (5/22/1019)	
20	Updated 301 Mission - Foundation Retrofit Drawings, (8/23/2019)	
21	Updated 301 Mission - Indicator Pile Program (6/19/2019), (7/19/2019), (8/23/2019)	
22	Project Manual Millennium Tower Perimeter Pile Upgrade, revisions through 8/23/2019	
23	Geotechnical Report_Rev 1 (Egan/Slate), including App A-E, 8/13/2019	
24	SGH Responses 7/23_2019 Perimeter Pile Upgrade EDRT	
25	Letter (7/25/2019) from SGH (Hamburger) to EDRT (Deierlein) regarding comments by Dr. R. Pyke	
26	Letter (7/10/2019) from L.B.Karp to the City & County of San Francisco Board of Supervisors	

Note to Readers: This comment log summarizes questions raised by the EDRT and responses from the design team over the course of the review of the permits for the 301 Mission Foundation Retrofit. The responses by the design team are often short summaries or notes to more detailed responses made during in person meetings along with supporting calculations and/or revisions to design drawings. The log includes a total of 191 questions (#'s in column 1), all of which have been resolved. In some cases, the comments and responses extend over several rows, reflecting back/forth exchanges between the EDRT and design team. The final resolution (last column) is listed once for each comment, even where the comment exchange spans over several rows.

Comment Log EXCEL filenames, as issued by EDRT

DATE	
1/16/2019	301 Mission Street - EDRT Comment Log V1
1/30/2019	301 Mission Street - EDRT Comment Log V2
3/12/2019	301 Mission Street - EDRT Comment Log V3
5/15/2019	301 Mission Street - EDRT Comment Log V4
6/26/2019	301 Mission Street - EDRT Comment Log V5
8/5/2019	301 Mission Street - EDRT Comment Log V6
8/26/2019	301 Mission Street - EDRT Comment Log Final

Comments by EDRT						Responses by Design Team			Date
1	1/14/2019	all	1	high	Please confirm the essential basis of the design (e.g., the basis and justification for the retrofit as a voluntary retrofit) as distinguished from other desired objectives (e.g., Calcs V1 - Design Overview (V1), pg. 2). Discuss how the vertical and lateral strength of the foundation system has changed as a result of the settlements, and confirm this should not be considered "Substantial Structural Damage". Confirm how the minimum building code requirements will be demonstrated, including applicable building code (e.g., SF-Existing BC 2016; ASCE 7-10; ACI 318-14) and any exceptions taken to the code (e.g., PEER TBI V2, ASCE 7-16 Chp 6).	ROH	2019_03_06	We confirm that damage to the building from settlement has not decreased the resistance by more than 10% and therefore, the upgrade is being performed as voluntary under the provisions of the San Francisco Existing Building Code, Section 403.9. Please refer to the updated Calculation Volume 1 for clarification of the design criteria. Also refer to Supplements No. 1 and No. 77	5/15/2019 - Resolved
2	1/14/2019	all	1,3		Confirm that the non-symmetric foundation retrofit does not create a plan irregularity (V1, pg. 4) and the effect of any eccentricity on the torsional building response and non-uniform foundation support (i.e., hard points created by installing new piles socketed within rock and variation of axial loads on the existing piles due to unloading on the north and west side of the mat).	ROH/LH	2019_01_30	We investigated whether the introduction of new piles along two sides of the structures creates plan irregularity as defined by the building code. Refer to the attached file "Comment 2". We modified our ETABS model described in Volume 3 of the permit submittal, Section 2, to include foundation flexibility associated with the pile lateral springs and representing the eccentric location of the new piles relative to the mat centroid. We compared displacements at opposite ends of the foundation due to ASCE 7-10 Section 12.8 Equivalent Lateral Force load cases including (+/-) 5% accidental mass eccentricity. We confirmed that the foundation retrofit does not result in plan irregularity as defined in ASCE 7-10 Table 12.3-1. Refer to Supplemental Calculation 02. Please also refer to our response to comment No. 132, where we indicate that the MCE rotation of the structure with the retrofit in place, is negligible and also, that the displacement of all existing piles is reduced relative ot the condition without the retrofit.. Note that this responds to an issue raised in Mr. Karp's letter of 7/10/19.	3/11/2019 - Resolved
3	1/14/2019	all	1,3,5	high	Confirm the MCE spectrum to be used as the basis of the MCE design evaluation and that it meets the SF building code requirements (e.g., 80% of ASCE 7-10, site class D), and the assumptions and details of ground motion selection and scaling for NLRHA.	JE/DM	2019_01_31	MCE spectrum is revised based on discussions during 21 Dec. 2018 meeting. Please see Supplemental Calculation 03	5/15/2019 - Resolved
4	1/14/2019	all	1		V1, Table 3-3: Please confirm: (1) if the foundation mat rotation criteria of 0.01 radians applies to the total rotation (including settlement deformations to date), total future rotation (future settlement plus MCE demand), or future earthquakes (MCE demand), and (2) the basis of the 0.01 radian limit, considering induced reinforcing bar strains.	ROH/LH	2019_01_30	1) The limit of 0.01 radians applies to the total value of plastic rotation including dead and live load, settlement to date, load from jacking of the retrofit piles, and MCE demands. Settlement analyses conducted by the Egan team predict that once the retrofit is installed there will be recovery of mat rotation and therefore, reduced demand due to settlement. 2) We adopted the 1% rotation limit as a lower-bound estimate of the ductility in the mat during our initial evaluation of the building. This value corresponds to the most conservative CP limits for RC beams given in ASCE 41-13 Table 10-7. We also performed a moment curvature analysis of the mat and calculated a maximum curvature of approximately 0.0005. Assuming a plastic hinge length equal to d this results in approximately 6% rotation capacity. Refer to file Comment 4 - Moment Curvature.xpj for the section analysis.	5/15/2019 - Resolved
	3/11/2019:				Clarification of the acceptance criteria is helpful, but the file for comment #4 was missing from the submitted materials.		2019_04_19	Please see file Comment 4 - Moment Curvature.xpj as part of the attached files.	
5	1/14/2019	all	1		V1, Table 3-3: Please confirm how the core wall rebar and wall strains will be determined in the MCE analysis as related to the justification for the specified acceptance criteria.	ROH/LH	2019_01_30	We obtained core wall tensile and compressive strains from Perform strain gauge elements located at the ends of wall boundaries. Strain gauges extend between adjacent floor levels and are one story high. Based on our review of high rise designs by other designers, this is consistent with current practice. We reduced concrete compressive strain acceptance criteria by a factor of 2 to account for findings by Wallace (2012) that Perform may underestimate strains by that much).	3/11/2019 - Resolved

Comments by EDRT						Responses by Design Team			Date
6	1/14/2019	all	1		V1, Section 3.6.2.1. Please confirm/edit the language that suggests the geotechnical analyses are not complete, e.g., "layering will include". We note further improvement to the geotechnical study is also referenced at the bottom of Page 12 / 46 of SLATE 11/30/2018 report.	ROH/LH	2019_04_19	We revised the language in Volume I of our calculations. Please refer to "Revision 3 - Volume 1 - Design Overview.PDF". The updated geotechnical report will also include revised description of the performed work.	5/15/2019 - Resolved
7	1/14/2019	all	5		Geo Report (Section 9.1, 9.2.3) - Some of the references to pile properties seem to be inconsistent with the latest design as described in the drawings (e.g., reference to 850 kip pile yield force, 18-in dia. steel casing, 3" dia. central bar). Please check and confirm that the geotechnical analysis reflects the proposed final pile foundation design.	JE/DM	2019_01_17	The pile design was updated after finalizing the current FLAC3D MC analysis. As the pile elements in the FLAC3D model were specified as elastic (i.e., no yielding behavior) and the loads were lower than the current yield force, we don't anticipate the conclusions and recommendations to be substantially different for the updated pile design. However, we do plan to incorporate the finalized pile foundation design into the FLAC3D model.	3/11/2019 - Resolved pending submission/review of results for the updated FLAC3D model. 8/5/2019 - Resolved
8	1/14/2019	all	5	high	Geo Report (Section 9.4) - Please confirm (1) what additional analyses are included with respect to the settlement analyses and soil-structure interaction (e.g., 3D settlement analyses and consideration of mat/superstructure stiffness in the settlement calculations), and (2) the timetable to complete the "future improvements" that will potentially influence the final foundation design. We note that settlement analysis performed to date is based on a decoupled analysis method (i.e., calculating the stresses within Old Bay clay using FLAC 3D and computing consolidation settlements using one-dimensional consolidation analysis method). If a decoupled analysis approach is adopted for final design, an adequate number of iterations between FLAC 3D and one-dimensional consolidation analyses has to be performed. The last iteration would ensure compatibility between stress calculations from FLAC 3D and one-dimensional consolidation settlement calculations. We note that the settlement values shown on Figure A-6 are the results of the first iteration.	JE/DM	2019_01_31 2019_03_06	~~To support the recommendations for the final design, we are implementing the iterative approach suggested by the EDRT for the decoupled analysis. That approach will be performed with sufficient iterations to attain reasonable compatibility between the FLAC3D stress computation and 1-D settlement calculation. That work is presently underway and we expected it to be accomplished within two to three months.~~ Upon further consideration of the iterative approach mentioned above, we would like to supersede the previously-submitted response with the following: The decoupled analysis of the expected settlement of the Tower due to secondary compression of the Old Bay Clay following the retrofit uses a two-step process. First, FLAC3D is used to compute the stresses beneath the Tower due to gravity loading, initially for the weight of the Tower and then the reduced stress associated with the load carried by the new Perimeter Piles. The resulting stresses are then extracted from FLAC3D and used as input into a spreadsheet to calculate the secondary compression settlement. This allows for more control over the soil constitutive law, as well as faster consideration of more scenarios (e.g., variability of soil properties). An iterative analysis is not possible in this scheme, as such an analysis would require that the input at the beginning of the analysis be the same as the final output. In this analysis, the "input" for the start of the analysis (i.e., in FLAC3D) is the building load and the "output" at the end of the analysis (i.e., from the spreadsheet) is the estimated settlement.	5/15/2019 - resolved pending review/acceptance of the updated geotechnical report and foundation performance evaluation 8/5/2019 - Resolved
9	1/14/2019	all	5	high	Geo Report (Sections 9.3 and 9.4) - Please confirm if the effect of tower tilting (e.g., eccentricity caused by tower tilting) is included in the FLAC3D analyses of soil stresses. If not, please include a justification to confirm what effect tilting will have on soil stresses and settlement.	JE/DM/CAR	2019_01_18	P-delta effects are not explicitly modeled in the FLAC3D analyses because they are small. Over the building's gross footprint, it imposes approximately 12ksf of pressure on the underlying soils, neglecting spread of load through the coma sands raft. The building currently tilts approximately 16 inches to the northwest at the roof, resulting in a displacement of the center of mass of the building by 5.6 inches (1/2 of 16 inches * .707) in each of the north and west directions. Assuming that the building and foundation behave as a rigid body, this eccentricity imposes about 720 psf additional stress at the northwest corner, due to P-delta effects, resulting in a 6% change in pressure relative ot the average uniform pressure without tilt. This is overwhelmed by differences in loading due to adjacent site conditions.	5/15/2019 - resolved pending review/acceptance of the updated geotechnical report and foundation performance evaluation
	3/11/2019				The difference in pressure between northwest and southeast corner is 1,400 psf which is more than 10% of average load of 12,000 psf. Please include in settlement calculation or provide further justification for ignoring the tilt in settlement calculations.		2019_04_19	We will incorporate consideration of this effect in our analyses of future settlement.	Comment closed. See new Comment 145.
10	1/14/2019	all	5	high	Effect of Future Consolidation on Integrity of the Existing Mat Foundation – Figure A-6 of the Geotechnical report shows the predicted settlement at the top of the Old Bay clay layer to have significant 'dishing' (distortion from a plane). How much of this distortion will be realized by the mat? Has mat stiffness and its effects on redistribution of settlement at depth been accounted for?	JE/DM	2019_02_04	The decoupled analysis accounts for the effect of the mat stiffness (as well as the stress relief from the Podium) on the stress distribution in FLAC3D, which is then applied in a Terzaghi consolidation analysis for each region of the Tower under consideration. We do note that analyses have shown that settlement calculated at the top of the OBC is a reasonable approximation of the settlement at the top of the mat in the first iteration of the decoupled analysis. Note that the additional settlement shown in Figure A-6 would be in addition to the current settlement; the current low spot is not in the same location as the low spot in the best estimate of additional settlement. Therefore, we anticipate the current dishing of the mat to be lessened after the retrofit is installed as the settlement on the east and south side of the Tower progresses.	8/26/2019 - Resolved
			23 and calcs.. In 2, 3	high	Please confirm whether the long term non-uniform settlement deformations (Geotechnical report Figure 6 and Section 8.1, page 9) will induce additional bending in the mat and whether these were considered in the mat evaluation. Calculation Volume 2, Section 1.2.6, suggests the measured displacements from June 2017 were used. Calculation Volume 3, Section 6, indicates settlement of the foundation was simulated, but was this the same approximate 2017 settlement, or future settlement (2060)?	ROH	2019_08_16	The mat has not been re-evaluated for the settlement contours predicted for 2060. Along a section extending east to west across the mat, where future settlements are predicted to be largest, the projected settlement results in an essentially rigid body rotation of the mat counter to the rotation that has occurred to the present date, which will tend to relieve some of the re-distribution that has occurred due to settlement to date. In a north-south aligned section through the area of most severe settlement, only very minor dishing is predicted which by observation, will not have significant impact on the mat.	
11	1/14/2019	all	5		Geo Report (Figure A-2, A-3) - Please provide further information on the data and analyses used to create the stress contours in Figures A-2 and A-3. Specifically, please plot preconsolidation pressure values obtained from laboratory testing on Figure A-3-B.	JE/DM/CAR	2019_02_01	Unit weights were estimated based on samples obtained during subsurface exploration at the Tower and adjacent sites. The hydrostatic groundwater level was estimated based on available piezometric data prior to and following construction of the Tower. The simplified preconsolidation pressure profile and variability was estimated by various consultants using laboratory testing performed on samples obtained during subsurface exploration at the Tower and adjacent sites. The initial estimated vertical effective stress profiles in Figures A-2A and A-3A (σ'_{v0}) were obtained from the initial equilibrated FLAC3D model (i.e., pre-construction). The current (2019) estimated vertical effective stress profiles in Figures A-2B and A-3B were obtained using the following procedure: [1) Simulate Tower and Podium construction in the FLAC3D model and allow the model to come to equilibrium, using best estimate soil parameters. 2) Extract vertical effective stresses from this model state. These effective stresses (σ'_{U100}) correspond to 100% degree of consolidation (the U=100% line in Figures A-2B and A-3B). 3) Where σ'_{U100} is less than the initial preconsolidation pressure, s'p0 (i.e., clay in recompression), assume that excess pore pressures are completely dissipated in 2019. This assumption is supported by performing time rate of consolidation calculations using ç in the recompression range. 4) Where σ'_{U100} exceeds σ'_{p0}, assume an average degree of consolidation of 85% (discussed more below), and calculate a "current" (2019) effective stress (σ'_{2019}) based on this degree of consolidation.] The estimated vertical effective stress profiles after the pile upgrade is installed (σ'_{ppu}) in Figure A-2C and A-3C were obtained by installing the perimeter pile upgrade in the FLAC3D model and allowing the model to come to equilibrium, using best estimate soil parameters.	3/11/2019 - Resolved
12	1/14/2019	all	5		Geo Report (Table A-3): Please provide further information on the test data to support the expected soil properties (and variability) in Table A-3. Additionally, identify which of the parameters are based on the total stress method (short term loading) and which parameters are based on the effective stress method (long term loading).	JE/DM	2019_01_31	All parameters are based on effective stresses. The average degree of consolidation in OBC (U) is based on analyses using 1D Terzaghi theory, Asaoka analyses of measured settlement data, fitting of time rate of consolidation curves to measured settlement data, and piezometer measurements of excess pore pressure. The change from best estimate preconsolidation pressure profile (P_m) is based on a statistical distribution of laboratory test data, where 2.64 is the standard error of the estimate. The multiplier on the stress change in the soil from Tower loads (S_m) is based on engineering judgement of a +/-15% variation. The compression ratio ($C_{c\varepsilon}$) variability is based on a statistical distribution of laboratory test data. The recompression ratio ($C_{r\varepsilon}$) variability is based on a statistical distribution of laboratory test data and correlation with $C_{c\varepsilon}$, where 0.01 is the standard error of the estimate. The secondary compression ratio ($C_{\alpha\varepsilon}/C_{c\varepsilon}$) is based on a statistical distribution of laboratory test data. The uniform groundwater drawdown in feet (d_d) is based on groundwater measurements at the site, and consolidation analyses to estimate the "effective" drawdown that caused effective stress change in the OBC. Plots are included as an attachment ("Consolidation Parameters.pdf").	3/11/2019 - Resolved
13	1/14/2019	all	5		Geo Report (Section 9.3.3 and Figures A-7 and A-8). Please confirm the criteria for discarding some of the MC simulations. The claim in the text (that simulations outside of +/- 10% were discarded) seems inconsistent with the histograms in the left panels of A-7 and A-8.	JE/DM	201_01_18	The criteria for discarding MC simulations outside of +-10% was applied to settlement calculations at SM-27 and SM-11 simultaneously; i.e., the settlement calculated at both locations must be within +-10% of the measurements in the same analysis to be "included". See Supplement No. 13 Monte Carlo Histograms.	8/5/2019 - Resolved
	3/11/2019				We think we understand the reasoning the elimination of certain points (i.e., saying that if the prediction is not on track for the 2018 current state, it can't possibly predict the 2059 future settlement well), however there is no mathematical (i.e., probability theory) justification for it. The simulations that are discounted don't look to be true outliers. In fact, including the discarded points provides a better prediction of 2018 settlement for SM-27, though it does appear to underestimate settlement at SM-11. While this approach may seem conservative (as the future settlement is larger when these points are eliminated), if the results don't install confidence in the model, the model should be changed (in this case the assumed distributions of the random variables), not results discarded.		2019_04_19	The analyses of estimated future settlement and associated Monte Carlo simulations are currently being revised and these criteria are being evaluated during that process.	
	5/15/2019				Please submit a description of the proposed approach for review.	JE	2019_06_07	This was submitted by email on 5/21 by John and discussed during the 5/22 meeting. It is also re-submitted now for reference as (Supplement 13 Slate_Memo_settlement_05-21-19.pdf).	

Comments by EDRT						Responses by Design Team			Date
14	1/14/2019	all	5		Geo Report (Pg. 10/46): Please confirm the assumed water table depths in the settlement calculations.	JE/DM/CAR	2019_02_01	The analysis assumed an initial groundwater table depth of 5 feet below ground surface. Figure A-3 shows the groundwater drawdown values used in the analyses.	3/11/2019 - Resolved
15	1/14/2019	all	8	high	Location of Test Pile: Please confirm the location of the test pile. The location shown on Drawing T001 (Beale Street) is different from the location we discussed at the project review meeting (corner of Mission and Fremont).	ROH	2019_01_30	An incorrect drawings set (from an earlier revision) was transmitted. Please see the updated drawings Supplement No 15	3/11/2019 - Resolved
16	1/14/2019	all	8		Evaluation of Slick Coat Friction Coefficient: We would suggest considering whether the pile test can be adapted to also evaluate the effective skin friction provided by the Slick Coat material (e.g., by installing another Osterberg cell above the rock and /or installing additional strain gages above the bedrock).	ROH/SKH	2019_02_02	We concur it would be valuable to obtain data on the effectiveness of load transfer throughout the pile length. Please see Supplement No. 16 for the proposed updated instrumentation plan	8/26/2019 - Resolved
	5/15/2019	all			Please confirm (1) the upper Osterberg cell is positioned sufficiently close to the base of the pile so as to fully develop the pile skin friction and end bearing in the rock (we would suggest it should be located closer, within ~5ft, of the base, and (2) the upper O-cell is positioned sufficiently close to the top of rock so as to overcome the pile skin friction with the rock so as to push the pile upwards through the clay and Colma sand layer.		2019_04_19	Please refer to the updated sheet T002 in Supplement No. 16-2.	
						SKH	2019_06_07	We presume that the reference in part (1) is to the lower Osterberg cell. The 8.5 ft. dimension indicated was based on the nominal O-cell load of 533 kips, the design rock strength of 12 kip/sf, and the socket area of 5.24 sf/ft. The O-cell can develop 650 kips, which can break the rock socket if the rock strength were as high as 14.5 ksf. If the O-cell cannot break the rock socket, the strength exceeds 14.5 ksf. Due to the RCD drilling technique, we do not expect to mobilize end bearing, as wet drilling spoils will remain at the bottom of the rock socket. With regard to (2) we are attempting to mobilize the portion of the pile between the O-cells and demonstrate the rock socket strength in a similar manner. We propose to include a third O-cell at the base of the Colma formation to verify that the portion of the pile above that elevation does not develop more than 80 kips total resistance (10% of the nominal pile load of 800 kips.	
	6/26/2019				The revised drawing T002 does not include the third o-cell to evaluate friction through the clay layer. Please either provide revised drawing or justification for not including the third o-cell.		2019_07_19	Pile design has been revised such that the annular space between the outer casing and 24" shaft liner will be filler only after jacking is performed. This eliminates the need for the third O cell.	
	8/5/2019			high	Page 13 of the Geotechnical Report notes that "Some amount of load transfer will occur within the soils above the bedrock; results of the planned indicator pile program will inform the extent to which load will be transferred to these soils." It seems that the 3rd o-cell is necessary to push the test pile up through the clay layer (above the rock) to determine the amount of force/stress transfer into the clay. [Note - we appreciate that it may be able to perform the push up test with the upper o-cell, i.e., by locking off the lower o-cell and relying on the friction and bearing over the lower 20 ft of the embedded shaft. However, this will require breaking the friction between the shaft and the 17 ft of rock above the o-cell, which may or may not be possible. So, the third o-cell would provide assurance that the test can be performed as planned.]	SKH	2019_08_05	We modified the indicator pile program (Drawing T002) to include a third O-Cell on , to be located 2 feet below the top of the upper Franciscan Formation. We computed the skin-friction capacity of the indicator pile above this point to be 521 kips, based on the presumed strength of the Old Bay Clay, the Alameda Formation, and the upper Franciscan, along with the reduction in skin friction provided by the SlickCoat. See also response to Comment 160. The Geotechnical report has been updated to capture this test procedure.	
17	1/14/2019	all	8		Strain Gage Locations: Please include the strain gage locations on Drawing T002.	ROH/SKH	2019_02_02	Please see response to Comment 16. Upon reaching concurrence we will update the drawing as requested.	8/26/2019 - Resolved
	5/15/2019	all			We suggest adding pair of strain gauges at interface of marine deposits and Colma sand so as to evaluate the friction of the pile through the Colma sand layer.		2019_04_19	Please refer to the updated sheet T002 in Supplement No. 16-2.	
						SKH	2019_06_07	We will include added strain gauges at the top and bottom of the Colma Sand layer, as well as continuous fiber-optic strain measurements throughout the length of the pile.	
18	1/14/2019	all	8	high	Please confirm and specify what steps to take and what instrumentation to provide during the pile test program to record drilling rates and other information to help establish criteria for adjusting the required rock socket lengths during future installation of the foundation piles.	ROH/SKH	2019_02_20	We concur that it is important to calibrate the drilling method against likely rock capacity. We propose to modify the specifications as follows: 1) Prior to the pile test a soil boring will be taken at the test pile location. 2) Drilling to be performed at constant crowd and torque, and drilling rate to be recorded during pile advancement, at each foot, to allow calibration against the boring data. 3) Take additional borings at intervals along the pile installation lines, to permit better forecast of probable rock conditions along the retrofit length.	8/26/2019 - Resolved
							2019_04_19	We updated our pile specifications to include the abovementioned modifications. Please refer to 31 63 29 Drilled Concrete Shafts 2019-04-09.PDF. A recommended approach for coordinating with DBI and EDRT will be included in the revised geotechnical report in response to comment 19.	
	5/15/2019				Section 3.04, C.1 - please complete the statement for the procedures for revising rock socket lengths, including (a) expected range of rock socket depths and (b) plan for reporting back to DBI and the EDRT any situations that deviate from the proposed procedure or depths that are significantly (to be specified) outside of the expected depths. Section 3.04E, we think the proposal to allow 72 hours between drilling/cleaning out the rock socket and placement of the rebar and concrete is too long. The rebar and concrete should be placed as soon as possible, preferably the same day but not longer than 24 hours later. Section 3.10 - both the GEOR and EDRT should be notified if any remedial action is taken, including (1) shortening of any rock socket by more than 10%, and (2) any significant shortening of more than 10% (5 out of 52) of the rock sockets.	SKH	2019_06_07	The procedure for modifying the rock socket lengths will be as follows: 1. Following review of the results of the O-cell tests in the indicator pile, we will revise the proposed depth of the rock sockets for the piles adjacent to the indicator pile location based on the interpreted average measured skin friction. 2. We will determine ratio of the measured skin friction to the tested unconfined compressive strength of the rock from the adjacent soil boring. 3. For the piles near to each of the subsequent six soil borings, we will estimate the rock-socket skin friction from the tested unconfined compressive strength of the rock and the ratio detmined in #2 above. We will adjust the rock socket length for each of these piles based on this computed value of skin friction. We have revised Paragraph 3.04 C.1 of the specification accordingly. We have revised Section 3.04E of the specification as requested in the comment. We will report all final rock socket depths to the EDRT and DBI. We have included this reporting in Section 3.09 E of the specification	
	8/5/2019	all	22	high	1) Shouldn't rock socket drilling rate be used as an indicator of local rock properties, in conjunction with other test data, to establish the rock socket length? If so, this should be included in the plan for interpreting the indicator pile data and making adjustments to rock socket lengths during construction of the permanent piles. 2) A report should be submitted to DBI and the EDRT at the end of the Indicator Pile Program (Section 3.02), with indicator pile installation and testing data, rock socket criteria for each production pile and any revisions to the pile construction procedure. 3) Related to the previous points, the specification seems to be missing a requirement for the contractor to log drilling parameters for both the indicator pile and the permanent foundation piles. For the indicator pile the drilling rate will be helpful to interpret the results. For the permanent piles, the drilling rate in the rock should be logged in real time (and submitted as part of the daily report) so differences can be identified and adjustments to the rock socket length can be made as soft spots are identified. 3) As part of the field quality control section of the specification the following points should be addressed: • The specification should include a requirement for submittal of daily reports, including what information the contractor tracks for each pile on the dailies. • The geotechnical engineer should provide continuous special inspection during drilling of the rock socket. Section 3.09C only lists a few items for periodic inspection, and it is unclear if they will be on site continuously during pile installation. It seems that someone representing the geotechnical engineer should be on site at all times during installation of the piles (requirements on S002 are more consistent with that approach). 4) Please confirm that the drilling and quality assurance requirements in the specification and on Sheet S002 are consistent. 5) In Section 3.09E, there should be a provision for timely notification and review by the Department of Building Inspection and the Engineering Design Review Team of any field adjustments that reduce the rock socket lengths by more than 10% from previously reviewed design lengths (i.e., design lengths originally determined based on the indicator pile data or subsequently adjusted and reviewed based on pile construction data).	**SKH**	**2019_08_05**	1) Yes, rock drilling rate will also be used as an indicator of the local rock properties. This is indicated in Paragraphs 3.02E and 3.03C. We included explicit requirements for logging of the drilling rate in the rock in new Subparagraph 3.03C1. 2) We included reporting requirements to the EDRT and DBI in the drawing notes on Sheet T002. See Drilled Piles Note 1d and Pile Testing Note 8. The Geotechnical Engineer will interpret the test results and will make recommendations regarding determination of rock socket lengths for the production piles. 3) See Item 2 above for indicator pile logging. We added requirements to the specification for logging the drilling rate, as indicated in Item 1 above 3') We included the requested additions to the Field Quality Control Article of the Specification in Paragraph 3.09E. We clarified Paragraph 3.09C to indicate continuous inspection of the pile drilling operations by the Geotechnical Engineer and during installation by the Special Inspector. 4) We modified the QA/QC requirements in the Specification and Drawing to ensure they are consistent. 5. We modified the last sentence of Section 3.09 E to read: "The Contractor shall facilitate the Geotechnical Engineer's and the Special Inspector's inspection of each drilled shaft. The Contractor shall not proceed with placement of reinforcing or concrete without approval of Geotechnical Engineer. The Geotechnical Engineer shall not approve placement, without review of the rock socket length by the EDRT and SFDBI, for two consecutive piles with rock socket lengths that are more than 10% less than design rock socket lengths."	

Comments by EDRT						Responses by Design Team			Date
19	1/14/2019	all	5,7, 12	high	Please include in the geotechnical report, the foundation retrofit drawings, and the specification: (1) the assumed design basis for the rock socket lengths, with provisions to update this based on the test pile, and (2) procedures and criteria for adjusting pile embedment lengths during construction based on drilling rates or other installation data. The procedures for making adjustments to the design pile lengths should include criteria for submittal of design and permit revisions with associated review and approval from DBI. We would recommend that when the adjustments exceed (a) the originally specified values by more than 10% at each pile or (b) affect more than 5 piles, DBI must be informed and review the adjustments.	JE/DM	2019_02_02	We will include in the updated/revised geotechnical report, specific recommendations regarding the rock socket design basis, provisions for updating the design based on results of the pile load test, procedures and criteria for selecting the rock socket depth and length characteristics based on observations of conditions encountered during installation and loading of the test pile, and procedures for review and approval by SFDBI of significant changes to the original design basis.	8/5/2019 - Resolved pending resolution of comment #18.
20	1/14/2019	all	5, 7	high	Please confirm how the presence of the existing shoring wall between the podium building and tower has been considered in the retrofit design and determination of future mat settlement. Specifically, will the gap between the mat and the top of the shoring wall close and result in additional tower tilt to the west, considering that settlement of the shoring wall is likely to be constrained by the podium building which does not settle at the same rate at the interface with the tower. Please compare the as measured gaps (from the LERA testing program) with the predicted future settlements from analysis.	JE/DM/CAR	2019_02_02	The existing shoring wall is included in the FLAC3D analysis and its contribution to the stress distribution is incorporated explicitly. The stress reduction at the interface between the Tower and the Podium after the PPU retrofit is about one-third of the stress reduction at the northeast corner (see Figure A-4). The calculated best estimate of the additional settlement at the interface between the Tower and the Podium after the PPU retrofit varies between zero and about two inches (see Figure A-6). This has the potential to close the gap between the mat and the top of the shoring wall; however, this will provide a load path for the Tower load to transfer directly to the OBC on the east side of the Tower. This will induce settlement on the east side of the Tower, which would ultimately increase the amount of tilt correction since the west side of the Tower will be "fixed" due to the PPU.	6/26/2019 - Resolved
	5/15/2019				3/12/2019: Geotechnical questions related to settlement are resolved. However, should the gap close due to continued settlement, the possible changes in the internal force distribution in the mat, wall and potentially the columns should be considered. Please confirm if these possible changes in force distributions are being considered. If so, then how, if not, they explain the basis for not evaluating these effects. 5/15/2019: Please check shear in the mat for subsections as discussed in the meeting on 4/23/2019, i.e., in 4 separate sections along the NS axes and in 3 separate sections along the EW axes. In addition, given the inelastic mat demands shown in the PG&E Vault in these and other analyses, please confirm whether the base mat of the PG&E vault has been inspected for damage (see new comment #125).		2019_04_19 **2019_06_07**	We performed an evaluation to see how the Tower and Podium foundations would respond, if continued settlement along the east side of the Tower were to bring the Tower mat to rest on the shoring wall along the Tower's east side and transfer vertical loads between the two foundation systems. We somewhat arbitrarily elected to evaluate the hypothetical case in which sufficient settlement occurs to transfer 80 kips per foot of vertical load from the Tower to the basement wall of the Podium through friction between the shoring wall and basement wall. This amounts to ½ the amount of load per foot transferred at the new perimeter piles, during jacking operations. We applied vertical point loads to our PERFORM-3D model at the east edge of the mat to simulate the bearing force we expect to develop at the wall. We applied these loads following application of the building's dead weight, settlement effects (simulated through thermal loading), and jacking of the new retrofit piles. We also checked the shear capacity of the podium mat and vertical capacity of the basement walls. We find all structures are adequate for this load case. See the attachment "Supplement No. 20" for more details. We have updated Supplement No. 20 for the requested mat subsections. See our response to Comment 125 for discussion of the PG&E vault.	
21	1/14/2019	all	7, 12	high	Drwgs S402, S502 and Specification: Specify appropriate corrosion protection for the thread rods that will transfer load into the piles.	ROH/SKH	2019_06_07	The threaded rods will be hot-dip galvanized over their full length. The portions of the rods that are within the PVC sleeves in the mat extension will be coated in corrosion-resistant grease. To confirm that the galvanized finish will not be subject to cracking under yield level straining, we conducted a series of tests of representative galvanized threaded rods in our Boston laboratory as discussed in the 5/22 EDRT meeting. One of these tests confirmed that the threaded rod has excellent stress-strain properties with a well defined yield plateau followed by hardening and ultimate strain of 21%, far in excess of that desired or needed. The second rod was tested in a fixture using nutted connections to a strain of 5% (the design strain level) and examined under microscope to confirm that the galvanized finish remained in good condition. A laboratory report is pending.	6/26/2019 - Resolved
22	1/14/2019	all	7		Drwgs: The EDRT suggests that the design includes provisions for access to the pile anchorage vault so as to permit regular inspection of the pile anchorages. The moisture within the vault should be carefully controlled by a designed system that may include elements such as a drain connecting to the basement drainage discharge system. The EDRT further suggests that the design allows for future adjustment to the jacking load on piles (which would also require access).	ROH/SKH/LH	2019_01_30	We concur with these comments. We have added manhole access to the vault, a sloping vault floor, and sumps, with "tell-tale" instrumentation. In addition, we have revised the jacking design to use a custom, mechanically locking system in lieu of the flat jacks. These jacks use a threaded ram, and a lock-nut, the nut locks the ram against the casing once desired jacking has occurred. This allows depressurization and repressurization for adjustment of the jack at a later time, if needed. The new jacks are somewhat larger (in height) than the prior jacks, and will require a somewhat taller vault, also facilitating access.	3/11/2019: Resolved
23	1/14/2019	all	7	high	Drwgs S402, S502: Specify how the bond/friction will be eliminated/reduced between the 24" pile casing and the 10' thick mat extension so as to permit preloading of the piles. Please provide supporting information to confirm the effectiveness of the bond/friction break.	ROH/SKH	2019_02_20	The detail will be revised to include an oversize sleeve (with 1/4" larger nominal inner diameter, and grease coating to assure slip between the pile and mat.	8/5/2019 - Resolved
							2019_04_19	We computed the expected lateral displacement of the proposed retrofit pile from the allowable out-of-plumbness tolerance using the upper-bound design settlement. We then analyzed the pile as fixed-head and computed the lateral and flexural forces that are developed at the head as a result of the imposed lateral deformation. We used these forces to compute the vertical friction force on the pile as a result of settlement. We computed a friction force of 11.3 kips which is insignificant when compared to the pile jacking force. Please refer to the attached "Supplement No. 23" for the results of our analyses.	
	5/15/2019				The calculations for supplement 23 (which refer to a friction calculation between steel and concrete, with the slip coat) seems inconsistent with the revised detail (per 2/20/2019) response to insert a steel sleeve through the mat extension with a greased surface between the pile casing and the steel sleeve. To the extent that the friction transfer is very sensitive to the shear and moment developed in the pile, the previously agreed upon greased sleeve detail is preferred. Please confirm the proposed detail.		2019_06_07	As discussed during the meeting of 5/22/2019, we will add a sono-tube sleeve to the detail, which will limit friction force at the mat-pile interface to levels below those we have estimated in Supplement 23. We revised the Supplement 23 friction force calculations to additionally account for pile head rotations induced by flexural deformation of the mat due to jacking and long term differential settlement. The anticipated friction forces without a sleeve (i.e. steel on concrete with slick-coat) are less than 9% of the jacking force.	
	6/26/2019				Please confirm the thickness of the sonotube material and provide an estimate of the expected pile movement (angular distortion) due to compression of the material.		2019_07_19	The thickness of the Sonotube material is 0.205 inches. This has negligible effect on overall response.	
24	1/14/2019	all	7	high	Drwgs S402, S502: Specify how the bond/friction will be eliminated/reduced between the 24" pile casing and the CLSM fill and soil below the mat so as to minimize loading of the underlying clay during pile preloading. Please provide supporting information to confirm the effectiveness of the bond/friction break. Has the variability in the friction coefficient been considered in the evaluation of stresses in the clay layers for settlement calculations?	ROH/SKH	2019_02_20	We have enclosed product data and a paper on the effectiveness of "slick coat" in reducing bond as Supplement No. 24. In addition, we will use a very low density material to fill the annular space. The effectiveness of this approach will be verified with the test pile.	8/26/2019 - Resolved
	3/11/2019				Thank you for the information on slick coat. Following up the second part of our comment, please confirm whether the variability in the friction coefficient has been considered in the evaluation of stresses in the clay layers for settlement calculations.	JE/DM	2019_04_19	Although we don't think further response beyond that above is necessary at this point, sensitivity analyses can be performed, if necessary, prior to the load testing to assess the effects of this potential variation for the estimates of future settlement following implementation of the Perimeter Pile Upgrade.	

Comments by EDRT						Responses by Design Team			Date
25	1/14/2019	all	3,7	high	Calcs V3 - Lateral (V3), Section 7.4: Has the RFI adjustment to mat shear reinforcement (V3, pg. 193) been considered in the evaluation of the mat shear capacity? Perhaps Vs should be neglected in the Vn term (V3, pg. 199); otherwise more analysis is needed to demonstrate that the headed bars will participate in a strut-tie mechanism for shear. Related to this - confirm if the representation of existing headed bars is correct in S401?	ROH/LH	2019_03_01	We evaluated the effectiveness of shear reinforcement in the mat foundation. We confirmed that pile dowel reinforcement extends sufficiently to lap splice the headed shear reinforcement in the mat and transfer its tension to the plane of the horizontal reinforcing bars. We reduced shear strength in the mat to consider only the area of headed bars which can be effectively spliced to pile dowels. Please refer to the attached "Supplement No. 25". The representation of existing headed bars will be updated on S401.	8/5/2019 : Resolved
	5/15/2019				5/15/2019 - As discussed during the meeting of 4/29/2019, given that a shear failure can potentially occur over a highly-stressed portion of the mat, please check the shear capacity of the slab in smaller segments (4 segments for sections cut along the NS direction and 3 segments for sections cut along the EW direction). In addition, please confirm how the shear demands used in this calculation compare to the demands that may develop if the mat settles and the new piles are loaded up to the yield load of the pile anchor (fuse) rods.		2019_06_07	1) We checked the shear capacity of the mat in smaller segments in our most recent analyses. Please refer to the May 2019 Revision 4 to the design calculation submittal for results. 2) Please refer to the results of our "push-down" analysis, in which we apply forces exceeding the yield axial capacity of the pile fuse. We have updated our push-down analysis to check shear capacities in the discussed sub-segments. Results of this analysis are summarized in Supplement No. 28.	
26	1/14/2019	all	4,7		Calcs V4 - Details (V4) and Drwg. S502: Please provide calculations to check the capacity of the anchorage plate attached at the base of the threaded anchor rods.	ROH/SKH	2019_02_04	Please see Supplemental calculation No. 26	5/15/2019 - Resolved
	3/11/2019				Plate size and submitted calculation seem reasonable. However, ASTM A970 for headed steel bars recommends use of a double nut on plate assemblies. It also discusses issues from bar straightness, roundness etc., recommending testing. Testing seems also desirable because bar size and material spec are specifically excluded in the ACI 318 for headed bars. Note that Williams has a spherical nut to deal with misalignment.		2019_04_19	We revised our details to include a double nut as well as a snug-tight jam nut above the plate that will prevent the plate from turning with respect to the axis of the rod. Using a spherical nut appears to be unnecessary given the use of a jam nut. Please refer to Supplement 26-2 for the revised detail. ASTM A970-13a does not include bar straightness requirements. The out-of-roundness evaluation referenced in ASTM A970 is related to thread dimensions and is applicable to reinforcing bars and not threaded rods. Adequacy of the threads in threaded rods is confirmed through manufacturer testing in accordance with ASTM F1554-18. We designed the assembly on 1/S502 using well-understood principles of physics and engineering mechanics. The materials and assemblies specified on our drawings conform to ASTM standards and are expected to exhibit predictable behavior. The testing prescribed in ASTM A970 is intended to verify the properties and behavior of headed bar assemblies that have manufacturer-specific plate material properties, plate dimensions, and threads. This is not the case in our detail as we use standard threaded rods and nuts. Therefore, testing of the assembly is not necessary.	
27	1/14/2019	all	2,7	high	Calcs V2 - Gravity (V2) and Drwg. S202: Has the influence of the shoring walls (for both the original mat construction and the retrofit) been considered in the response under lateral earthquake effects, including possible effects on pile deformations and/or causing an eccentricity in foundation response?	ROH/LH/JE	2019_01_30	Our models have not considered the additional strength and stiffness of either the original shoring walls or the new shoring walls. We believe this approach is conservative in that it requires a greater percentage of the total load to be resisted by the piles.	3/11/2019 - OK, Resolved
28	1/14/2019	all	3,4		Confirm what pile forces are used in the analysis and calculations to check the mat shear/moment capacity and the connection to the new mat extension (Design 800 kips? Thread bar strength 1113 kips? Or Other?)	ROH/LH	2019_01_30	We used the design jacking load of 800 kips in combination with Dead, Live, Settlement and MCE shaking to check the mat capacity in Perform-3d and SAFE. For design of the connection to the new mat extension we increased this load by a factor of 1.5 in order to prevent yielding at the cold joint between new and existing concrete. We also performed a push down analysis of the mat-pile system, to confirm that the mat and extension are capable of developing a full plastic pile capacity of 1500 kip	8/5/2019 - Resolved
	3/11/2019				The thread bar strength is calculated based on 75ksi and the full bar area. In reality, the tensile strength could reach 95ksi (allowed range per spec), and the stress area of the threaded bar may be less than the 3.98 in2 used in calculations. The bar strength can exceed 1,113 kips, and can reach 1,236 to 1,512 kips (per ASTM F1554). Calculations show demands based on the Factor of Safety of 1.5 on the 800 kip jacking load. This equates to 1,200kips, which is less than the 1,500 kips the rods can potentially develop. The impact of the higher load should be evaluated. The push down analysis does not appear to be documented in the submitted calc package.		2019_04_19	Please refer to Supplement No. 28 - Pushup Analysis Summary.PDF.	
	5/15/2019				Thank you for providing the pushdown analysis in Supplement 28, but we have several questions about the results: 1) Please confirm whey the yield moment and curvature are much smaller for beam 2 (Figure 12) as compared to beams 1 and 3 (Figures 11 and 13) -- also not that Figure 13 appears to be mis-labeled? 2) Please confirm the tributary widths of the mat used to determine the DCRs in Table 1 (this is related to other questions about the mat shear and whether it is being checked local or across the full mat width). 3) Please confirm whether checks of the moment and shear strengths of the mat extension (and particularly the extension-mat interface) are addressed in these calculations.		2019_06_10	1) Please refer to Figure 1-24 in Vol 2 of our May 2019 revision to the permit submittal. Beams 1 and 3 are located in the region of increased reinforcement at the outriggers (H10), with positive moment capacity of about 21,500 kip-ft. Beam 2 is located in the region with relatively smaller area of bottom reinforcing steel (4-H2), with positive moment capacity of about 11,000 kip-ft. Please note that the second point on each curve corresponds to Step 2 of the pushover analysis, and does not show the element rotation at yield. The plots show only inelastic rotations, which is why each beam has a different initial moment at zero rotation. Yield of each beam occurs at a jacking force somewhere between those applied at Steps 1 and 2. Curvature at Step 2 is smaller for Beam 2 because gravity demands are much higher in Beams 1 and 3, which are located near the outriggers. We have corrected the Figure 13 label in our update to Supplement No. 28. 2) We re-ran the pushdown analysis with smaller section cuts consistent with the subsections we have discussed (3 EW and 4 NS). Please refer to the updated Supplement No. 28 for results. 3) The calculations in Supplement No. 28 account for the flexural strength of the mat extension which was modeled using nonlinear grillage beams. We did not check shear at the mat extension in these analyses. Shear was checked in the existing mat at the section cuts noted in Figure 8. Shear at the mat extension interface with the existing mat typically has low DCRs in the range of 0.4-0.6 for a pile vertical load of 1200 kips. SAFE Design Strip 1 is a short east-west design strip located just north of the PG&E vault along Front Street. SAFE Design Strip 1 is the only strip with a higher shear DCR which equals 0.91. Therefore, we do not anticipate shear failure at the mat extension interface to occur in the pushup analysis. Shear design for the mat extension are available in Volume 4 of our calculations.	
	6/26/2019				Thank you for the updated supplement (6/7/2019). Please provide calculations to confirm how the expected shear capacities are calculated for sections SC1 and SC6 in Table 1.		2019_07_19	Detailed calculations of the expected shear capacities are included in Supplement 28.	
29	1/14/2019	all	7,12		Construction Monitoring: What are the plans for monitoring the building settlement (or other response parameters) during the construction and pile prestressing?	ROH/JE	2019_01_31 2019_03_06	During the construction and for at least three to six months after completion (depending on what the measurements are showing), we anticipate that we would continue the weekly surveying and interpretation of benchmarks located on the mat and of prisms mounted on the building façade. After that timeframe, we expect that the frequency of the monitoring can be decreased to monthly for two years, possibly three, and then quarterly through year five or six, and thereafter, annually would be adequate. These timeframes can be adjusted, depending on what the measurements are showing. We expect the changes from one measurement time to the next will be quite small, so it may be appropriate to move to longer measurement-frequency timeframes more quickly as we go forward after the initial period following construction. Please also see our response to Comment No. 091.	8/5/2019 - Resolved
	3/11/2019				The proposed short to longer-term monitoring sounds reasonable, but we need to develop more specific information on (1) specific measurements and estimated bounds of performance, and (2) procedures and requirements on reporting information back to DBI. In addition to the long-term monitoring, further thought and information should be developed regarding the detailed monitoring and observations before, during and shortly following the pile stressing operations. In particular, there should be estimates on the following information during pile stressing, (1) pile movement, (2) deformations of the mat, and other tell-tale response measures. All of this information should be written down and specified in the construction drawings and specifications.		2019_04_19	An updated, more detailed description/schedule for the geotechnical aspects of the monitoring was provided to the EDRT via email on March 22, 2019. We are preparing a drawing that will show all required monitoring during and after construction to include piezometers, inclinometers, pile load cells, mat vertical displacements and survey points located on the building facade.	

Comments by EDRT						Responses by Design Team			Date
	6/22/2019				We have a few comments regarding the proposed monitoring program as outlined on the new drawings S204 and S205. (1) On comment #7 on S204, is the intent to require 24 hours between each of the 100 kip load increments (e.g., requiring 8 days for loading up to 800 kips)? Or, is the 24 hours the maximum permissible time between load increments? (2) Please confirm and clarify comment #7. Please confirm and clarify on the drawings whether the intent is to require that the EOR be on site during pile loading, and if not, then clarify the actions that the contractor should take if the mat deflections deviate from the analysis predictions (within a reasonable tolerance). (3) Can you provide an estimate of the expected time (project delay) for the soil rebound and secondary loading, as required by comment #9 on S204.		2019_07_19	1) Confirmed. The intent is to require 24 hours between each of the 100-kip load increments. 2) Please refer to Note 4 under Statement of Structural Observation on sheet S002. The intent is for jacking operations to be observed by the Structural Engineer. 3) The project geotechnical engineer estimates that approximately 1 inch of rebound will occur over a period of about 2 years. Note that this calculation neglects the reduction in jacking load that will accompany rebound and therefore, is likely an upper bound estimate on the amount of rebound. We will monitor pile loading (through load cells) over time, and make judgements as to whether re-jacking is necessary.	
30	1/14/2019	all	6,12		In the shoring drawings and specification, please indicate the type and amount of dewatering that is expected to be necessary for the excavation and construction of the foundation retrofit.	ROH/SXY	2019_02_02	No dewatering is planned as part of this project. Based on recent investigations, the present water table is at approximately 22 feet below grade and therefore, the hydrostatic pressures are modest. We plan to control water infiltration into the excavation through a combination of jet grouting and chemical injection We do not anticipate any significant lowering of the water table associated with this work. It is likely that some minor water infiltration will still occur. We will modify the specification to require that the Contractor maintain sumps and pumps as needed to maintain the excavation sufficiently dry. Note that this comment responds directly to one of the points raised in Mr. Karp's letter of 7/10/19.	8/5/2019 - Resolved
31	1/14/2019	all	9		Drawings C2.1 and C3: Please confirm whether the proposed pile locations have been coordinated with utilities to ensure that the configuration of new piles will work with the utilities that are to remain in place (e.g., high pressure water on west and north faces)	ROH/LH	2019_02_02	We coordinated the pile locations with utilities that will remain in place. Please refer to the attached supplemental sketch Supplement No. 31, showing proposed locations of the piles. We will revise the drawings to incorporate this revised spacing	3/11/2019: The reference supplement appears to be mis-numbered (supplement #30 applies to the pile layout). Resolved pending submission and review of the final pile layouts. 8/26/2019 - Resolved
32	1/14/2019	all	6		Drwg. S101 - Correct numbering of the construction steps (i.e., change step 7 to step 6/7).	ROH/SXY	2019_02_02	Drawings will be revised to correct this.	3/11/2019: Resolved pending submission of the final drawings. 8/26/2019 - Resolved
33	1/14/2019	all	4	high	Interaction between tower and podium: Please confirm how the interaction between the tower and podium has been considered in the MCE analysis and by how much the seismic response of each building is affected by the interaction.	ROH/LH	2019_02_02	We modeled the interaction between the tower and mid-rise explicitly with gap elements and elastic links which approximately represent the stiffness of the load path through the at-grade and below-grade diaphragms of the podium. The podium was idealized as a SDOF system. We computed the podium base shears with and without the retrofit piles. They are essentially the same. Refer to Supplement No. 33 for additional details.	5/15/2019 - Resolved pending submission/review of the revised/final building analysis results. 6/26/2019 - Resolved
	3/11/2019				Please confirm how the induced base shear forces in the podium building compare to the design base shears for the podium building. Have any checks been made to confirm the safety (integrity of the shear walls) in the podium building under the induced forces?		2019_04_19	We compared the design base shear for the podium building to the demands from our analyses. The peak shear from our analyses with the MCE ground motions is approximately three times the design force with this elastic model of the podium building. We believe this is not unrealistic given that ASCE 41 would permit m values ranging from 4 to 6 for walls of this type. Regardless, we plan to model the podium building as an elasto-perfectly plastic SDOF stick with expected yield strength and strain hardening behavior to assess its global seismic performance. Refer to the attachment “Supplement No. 33-2” for plots of base shear demand vs time obtained from our current analyses.	
34	1/14/2019	all	1,5		Please confirm the basis for assuming Site Class D for the design, given that some layers of the underlying soils have low shear wave velocities with Vs(30) below the minimum value (600 ft/sec) specified in ASCE 7-10 for Site Class D w/o accounting for presence of piles, i.e. please demonstrate that presence of piles would increase Vs(30) to a value greater than 600 ft/s.	JE/DM	2019_02_02	The comment seems to be aimed at understanding why Site Class D has been selected instead of Site Class E. Section 20.3.2 of ASCE 7-10 and 7-16, Soft Clay Site Class E, states "Where a site does not qualify under the criteria for Site Class F and there is a total thickness of soft clay greater than 10 ft (3 m) where a soft clay layer is defined by Su < 500 psf (Su < 25 kPa), w >= 40%, and PI > 20%, it shall be classified as Site Class E." Ground motions for the structural analyses of the Millennium Tower are being applied at the elevation of the base of the Tower mat, so approximately 25 feet bgs. As such, to estimate V_{S100} (aka V_{S30}) and Site Class appropriate to characterizing map-based seismic parameters for comparing Code requirements to the site-specific ground motions, we have considered the soil profile extending 100 feet from the bottom of the mat. As indicated by the comment, there are some zones within the subsurface profile that have shear wave velocities (V_S) less than 600 fps, notably within the Recent Bay sediments (i.e., Young Bay Mud and Marine Sands). This is illustrated by the attached figure of shear wave velocity measurement obtained using suspension logging in Boring TTB-08, drilled on Fremont Street within about 25 feet of the Tower. We note that there are two zones, each approximately 5-ft-thick, for which the shear wave velocities (V_S) fall below 600 fps. We note however, that even with these thin low-velocity zones, the V_{S100} for the profile exceeds 600 fps and falls within the V_{S100} range (600 to 1200 fps) for Site Class D. In addition, examination of the measured undrained shear strengths (S_u) for the soils within the profile (see attached figure) indicates that none of the laboratory- or field-measured undrained shear strengths within the profile below the bottom of the mat are less than S_u=500 psf and the average S_{u100} is ≈ 1000 psf. Given that the calculated V_{S100} for the profile exceeds 600 fps, none of the measured undrained shear strengths are less than 500 psf (a requirement of ASCE 7-10 Section 20.3.2 for designating a site as Soft Clay Site Class E), the average S_{u100} is ≈ 1000 psf, and the average SPT penetration resistance for the cohesionless soils, N_{ch100} is ≈ 40, we are of the opinion that Site Class D is the appropriate Site Class designation for the soil profile. See Supplement No. 34 for visual comparisons. Lastly, we also note that CSMIP designates the profile as Site Class D for the strong motion station (CGS # 58411) installed at the Tower.	5/15/2019 - Resolved pending submission/review of the revised/final geotechnical report 8/26/2019 - Resolved
	3/11/2019				Before closing out this comment, please confirm how the presence of the existing piles factor into the effective shear wave velocity for the site and whether the effect of the piles can be confirmed by the FLAC3D analyses.		2019_04_19	Based on discussions at the March 14, 2019, meeting with the EDRT, this comment was stated by the EDRT as being resolved with no further action required. Although as agreed during the discussions mentioned above, consideration of the presence of the existing piles is not necessary to characterize the site with a Site Class D designation, at the request of the EDRT, we will add some commentary in the report addressing this topic.	
35	1/30/2019	SV	5		Section 9.2.1 & Table A1: For OBC, an elastic model of consolidation has been used. Please justify Elastic Modulus for this layer presented on Table A1	JE	2019_03_01	The elastic (constrained) modulus was updated based on the following relationship using a FISH function in FLAC: E = (1+ν) * (1-2*ν) * σ'v * ln(10) / Cxε / (1-ν) where Cxε = Crε in recompression ($\sigma'v < \sigma'p$) and Cxε = Ccε in compression ($\sigma'v < \sigma'p$). This method of computing the elastic (constrained) modulus was used to accommodate requests from the original expert panel and to reduce the calibration time necessary for a more advanced constitutive model. Text added to Section 9.2.1 (now Section 10.2.2).	5/15/2019 - Resolved
36	1/30/2019	SV	5	Moderate	Section 9.2.3 & 11.7: No load zone has been specified as 50 feet below the surface of OBC; whereas, in section 11.7 and during the meeting on 12/21/2018, it was stated that all axial pile loads will be transferred to bedrock. Please clarify.	JE	2019_03_01	Section 9.2.3 describes preliminary analysis developed as a proof of concept. An alternative retrofit proposal assumed that all the load on the retrofit shafts would be transmitted to bedrock; we initially used a similar concept in the preliminary analysis but updated the geotechnical model to include the no load zone only in the upper 50 feet of the OBC.	5/15/2019 - Resolved
37	1/30/2019	SV	5		Section 9.2.5 :Please provide site-specific relationships for secondary consolidation based on laboratory testing performed.	JE	2019_03_01	Per the text in the report, the laboratory testing is ongoing. We will provide this information once the laboratory testing is complete. We anticipate completion of this testing (which initiated more than a year ago) in a few months.	8/5/2019 : Resolved

Comments by EDRT						Responses by Design Team			Date
38	1/30/2019	SV	5		Table A3 : Please justify degree of consolidation of 0.85. Does using this value result in observed settlement to date? Logically, degree of consolidation varies within OBC layer. Why a uniform shape been assumed for degree of consolidation? Please justify using variation in preconsolidation pressure of -2.64 and +3.96. Why best estimate recompression index (0.1) is outside of minimum to maximum range?	JE	2019_03_01	The degree of consolidation is based on several factors: [1) Measurements of excess pore pressure (difference in observed water level versus hydrostatic) from piezometers installed in the OBC near the Tower, compared to increases in vertical stress caused by the Tower. 2) Consolidation analyses using 1D Terzaghi theory. 3) Asaoka analyses of measured settlement data. 4) Fitting time rate of consolidation curves to measured settlement data.] This estimate of the degree of consolidation results in calculated settlements that approximately match the observed settlement when used in concert with reasonable values of the other relevant parameters. We understand that degree of consolidation varies with the OBC layer; the assessment is based on comparing the field data to the normalized isochrones from a traditional 1D Terzaghi-type consolidation analysis. There is no mention of a uniform "shape" for the degree of consolidation; we assume that this question refers to the "distribution" column in Table A-3, this refers to the statistical distribution of the value as used in the Monte Carlo analysis. Figure 1 in Supplement 12 (previously provided) shows the justification for the preconsolidation pressure statistical characterization. The recompression ratio is characterized by the equation Cre = 0.12Cce+0.01X, where X is a random variable with a uniform probability distribution from -1 to 2. The "best estimate" value in Table A-3 is for the random variable X, not Cre. Following the equation, and using the best estimate value for Cce, the best estimate value for Cre is 0.035. Figure 4 in Supplement 12 shows the statistical distribution of Cre.	5/15/2019: Resolved.
39	1/30/2019	SV	5	High	Section 9.2.6 : Why calculated settlement at the surface of OBC has been compared with settlement markers at the surface of mat? This assumes OBC settlements are directly transferred to the mat; this assumption ignores mat's rigidity.	JE	2019_03_01	We incorporated structural elements representing the mat's strength and stiffness characteristics in the FLAC3D mode. We then performed an analysis case in which we applied gravity loads from the Tower to the top of the mat and calculated vertical displacements within the underlying soil profile and at the top of the mat. In a second case, we imposed displacements at the top of the OBC and compared them to the displacements at the surface of the mat. In both cases, the general magnitude and distribution of the displacements at the top of the OBC and the surface of the mat, although not identical, were similar. As a result, we concluded that displacement at the top of the OBC is a reasonable approximation of the displacement of the mat.	6/26/2019 - Resolved
	5/15/2019				Please confirm that the displacement at the top of the OBC is consistent with the mat displacement (tilting and slight dishing), by confirming the assumed deformed shape of the mat over time as discussed in 3/22/2019 meeting (i.e., demonstrating that most/all of the deformations since construction of the core wall and outriggers has been rigid body settlement and tilting)		2019_06_07	Supplement No. 39 is attached. The plot shows differential contours between 05-11-2011 and 03-21-2019 which demonstrate rigid body movements.	
40	1/30/2019	SV	5		Section 9.4 : In the statement " … the proposed perimeter pile upgrade would be designed to withstand additional settlement as much as 12 inches.", please identify where 12 inches has been assumed to occur.	ROH	2019_03_01	The 12 inch allowance was set early in the design process, before detailed analyses were conducted, to conservatively provide for residual settlement of the Old Bay Clays under secondary compression following the installation of the upgrade. The present structural design provides for 8 inches of post-jacking settlement based on more reliable analyses that this settlement will be much less. Please refer to Attach A in the Geotechnical Report. The 12 inches was assumed to occur within the OBCs beneath the Marine Sands and it was also assumed, this would result in similar mat settlement.	5/15/2019: Resolved.
41	1/30/2019	SV	5		Section 10.3 : Passive resistance on the south side has been ignored. Please demonstrate that this is not an overly conservative assumption. Also it would be appropriate to capture maximum soil resistance on the south side and compare it with the maximum passive soil pressure the train box has been designed for.	ROH	2019_03_01	Indeed, we believe that neglecting passive resistance against the south side of the mat is a conservative assumption and results in prediction of pile displacements and demands that are unrealistically large. This very conservative assumption shows adequate performance. With regard to the adequacy of the TTC structure to resist soil stresses associated with seismic response of the tower, we understand the TTC was designed to accommodate this as part of its original design, since the 301 Mission project was already in place at that time. Nothing we have done as part of the retrofit will increase these pressures. Therefore, we are not concerned about performance of the TTC train box.	5/15/2019: Resolved.
42	1/30/2019	SV	5		Section 10.4 : It is agreed that due to silo effects, the at-rest and active soil pressure on the south side would be minimal. However, train box would interact with the soil in front of south side, and as such, it would be prudent to account for the interaction between the two structures.	ROH	2019_03_01	As noted in our prior response, we believe that neglecting the train box and its interaction with the Tower is conservative. Given the complexity of including this structure in our modeling, we believe the added complexity of engaging in such modeling is not necessary for design of the perimeter pile upgrade.	5/15/2019: Resolved.
43	1/30/2019	SV	5		Section 10.4 : Please confirm 1 inch gap exists on the east side between the tower and the podium. Other values for the gap have been referred to in various reports. Furthermore, it is not clear how various lateral resistances are combined as presented on Figure B-3. Please provide sketches showing the tower base wall and mat thicknesses / lengths in all four directions for the tower and for the podium.	JE/ROH	2019_03_01	In August 2017, a series of cores were taken between the garage west basement wall from north to south and tower mat, to evaluate whether the mat had come to rest on the shoring wall. Our evaluation of the cores confirms that approximately 1 inch of foam is present in this gap. The lateral resistances in Figure B-2 (in units of kip/(unit length in feet) vs inches of deflection for 5 foot increments of wall height) are combined by multiplying the width of the wall by the values on the curve, then adding the resulting values over the depth of the wall at given wall displacements. This is repeated separately for each wall in question. The load reduction during the transition from the at-rest state to the active state is done similarly, except the load is reduced instead of increased. Supplement No. 43 includes clarifying sketches.	5/15/2019: Resolved.
44	1/30/2019	SV	5	High	Section 11.1 : It is stated that "the axial pile resistance will be modeled as a single spring in the vertical direction in the structural model". Please clarify how changes in axial pile loads during static loading condition (due to tilting of the tower) and during design seismic event will be accounted for using a single spring representing axial pile group stiffness.	ROH	2019_03_01	Vertical pile resistance in the structural model is simulated using nonlinear springs, limited by soil-pile bond. Evaluations of pile structural capacity confirm that structural capacity exceeds likely geotechnical capacity. Pile springs are provided in a distribution beneath the mat that is similar to that present in the actual structure, but adjusted to accommodate the nodal point locations in the gridded "mat" representation in the model. Pile stiffness have been adjusted to account for the difference in actual piles/per square foot of mat, and that included in the model. Tilting of the mat, due to long term settlement has been represented through a complex scheme of nonlinear elements (gaps and hooks) as well as thermal loading (simulation shortening of some piles) so as to reproduce the measured mat contours. At each pile, two different vertical spring representations are used. The first, simulates long term gravity conditions + settlement + jacking. The second spring is activated for seismic loads. Please refer to calculation Volume 2 Section 1.2.6.2 for details.	5/15/2019: Resolved.
45	1/30/2019	SV	5		Section 11.3 : What is the best estimate of actual axial pile loads with and w/o consideration for tilting? We note that Figure C-10 provides ratio of individual pile loads to the average pile load, but average pile load hasn't been specified.	JE	2019_03_01	The best estimate of actual axial pile loads considering non-uniform settlement but not P-delta (eccentric load) effects are shown on Figure C-10. As in the response to Comment 9, P-delta effects from the tilt of the Tower are negligible with regard to individual pile load demands. The average pile load is approximately 230 kips.	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
46	1/30/2019	SV	5	High	Section 11.6 : Please comment on computed existing pile head rotations shown on Figures C-11 and C-12 with those estimated by LERA.	ROH	2019_01_13	We do not know precisely how LERA computed their pile head rotations, though based on some intermediate results they presented, it appears that in some analyses they imposed a uniform rotation on the piles heads equal to the planar tilt of the mat. In our calculations we imposed unique rotations on each pile, computed based on the north-south and east-west tangent slopes of the mat computed using smoothed contours fitted to the 38 measured settlement markers. The result of our computations is that some piles exhibit reversed rotation to that suggested by the uniform mat tile approach.	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
47	1/30/2019	SV	5	high	Section 11.5.4 : The recommended ultimate skin friction in Franciscan Formation of 12 ksf is somewhat higher than those recently measured at Oceanwide project site which was in the range of 8.5 to 9.0 ksf. Also, for 1,500 kips load, the rock socket length is reported as 32 feet which is in conflict with 12 ksf ultimate frictional capacity. Please clarify.	JE	2019_03_01	Ray et al. (2018) report the results of five Osterberg Cell (O-cell) load tests conducted in shale mélange at elevations between -259 ft and -318 ft within Franciscan Complex bedrock in two drilled shafts at the Oceanwide project site. The mobilized ultimate unit side friction values from those tests were 7.3 ksf, two at 9 ksf, 9.4 ksf, and 13.1 ksf. Similar testing in sheared shale interbedded with greywacke sandstone at elevations between -245 ft and -264 ft in a drilled shaft at 706 Mission Street produced mobilized ultimate unit side friction values of 12.1 ksf and 13.1 ksf (Langan, 2017). For a tension pile load test conducted in sheared shale and greywacke sandstone mélange at elevations between -241 ft and -251 ft in a drilled shaft along Beale Street at the 301 Mission Street site, an ultimate unit side friction value of 18.4 ksf was mobilized. These load test results certainly indicate there is a range of ultimate unit side friction possible for Franciscan Complex mélange similar to the "sheared shale supporting blocks of wacke" encountered in borings at the Tower site, but that an ultimate unit side friction of about 12 ksf is reasonable as a central tendency or median value. The proposed diameter of the rock socket for the new Perimeter Piles is 20 inches, corresponding to a circumference of about 5¼ ft. In the report, we indicate that we are currently assuming an ultimate unit side friction of 3 ksf for the upper 10 feet of the bedrock. Given that, the contribution to the ultimate capacity of this portion of the rock socket is Qi = (5¼ ft)*(10 ft)*(3 ksf)≈ 160 kips. The remainder of the 1500 kips ultimate capacity must then be derived from the deeper portion of the rock socket. The necessary length of that deeper portion would thus be given by L = (1500 kips -160 kips)/((5¼ ft)*(12 ksf)) = 21+ ft, so≈ 22 ft. As such, for these assumed ultimate unit side friction values, the upper and deeper portions of the rock socket would need to total together about 32 ft. Regardless, the on-site pile test to be conducted as part of this project will confirm effective available skin friction and we will adjust required pile length once that data is available.	5/15/2019: Resolved pending review of the final review of the pile load test results.
48	1/30/2019	SV	1,2		Section 1 of Volume 2 refers to ASCE-41-13; whereas, ASCE41-13 is not referenced in Volume 1.	ROH/LH	2019_03_01	Concur. ASCE 41-13 was added to Section 3.1 of Volume 1. Please refer to the attached updated Revision 2 - Volume 1.	5/15/2019: Resolved

Comments by EDRT						Responses by Design Team			Date
49	1/30/2019	SV	2		Section 1.1.2 : Water is 10 feet bgs and not 3 feet. Please confirm/clarify.	ROH/LH	2019_03_01	Geotechnical computations were performed assuming 10 feet. For structural analysis, we assumed a depth of 3 feet bgs based on the 13 January 2005 Treadwell & Rollo report high groundwater elevation estimate. The report actually states that the high groundwater elevation is at -3 feet relative to the SF City datum which is approximately 6 feet bgs. We adopted this assumption in the early phases of the project in an attempt to simulate a conservative stress condition for the mat. By lowering the support springs that represent the existing piles using temperature loads in certain areas the springs would unload completely and hydrostatic uplift would cause increased out-of-plane stresses in the mat. In constructing the lateral backbone for the piles we use the pile axial loads provided by Egan. This assumption therefore only affects the stresses in the mat which we believe are conservatively bounded.	5/15/2019: Resolved
50	1/30/2019	SV	2	High	Figure 1-27 : Please provide a comparison between LERA's estimated existing pile cap rotation (maximum reported rotation of 0.54 degrees or 0.009 rad) and those estimated by SGH by plotting both sets on this figure. Explain the difference.	ROH/LH	2019_03_01	Figure 1-27 of Volume 2 is not directly comparable with LERA predictions of rotation. Figure 1-27 shows the nonlinear moment-rotation relationship we adopted for grillage elements in the mat. LERA has not indicated what assumptions, if any, they used in this regard. The 0.54-degree rotation about the X-axis shown in LERA's documents is not at the section level, but rather cumulative for the mat as a whole and includes rigid body rotation, and mat deformation. Similar plots of rotation about the X and Y axes from our model are shown in Volume 3 Figure 5-18 of the calculations. Volume 3 Figure 6-33 shows the plastic rotation values from our analysis as a fractions of 1%. .	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
52	1/30/2019	SV	2		Figure 1-27: Please provide strain in mat concrete and rebar corresponding to mat rotations presented on this figure.	ROH/LH	2019_03_01	Using our XTRACT fiber model developed for Comment 4 at a mat plastic rotation of 1%, which is approximately equivalent to a total section curvature of 0.00012/inch, we get maximum concrete strain of 0.0008 and maximum tensile reinforcement strain of 0.014.	5/15/2019: Resolved
53	1/30/2019	SV	2		Figures 1-32 & 1-33 : Has the effect of existing tilting of the tower been captured on the results shown on these figures?	ROH/LH	2019_03_01	Yes. The effect of tilt was reproduced by assigning a set of displacement loads to the support springs in the model. The intent of the displacement loads was to result in a mat deformed shape in the model that is similar to the shape obtained from subtracting the first mat survey data from 2007 from the most current survey data at the same points.	5/15/2019: Resolved
54	1/30/2019	SV	3		Section 5.2 & Figure 5-12 : Please state at what pile-head displacement the shear capacity of piles are reached, i.e. would brittle pile failure in shear occur before or after the pile experiences ductile failure in bending?	ROH/LH	2019_03_01	The existing precast piles were detailed in accordance with the requirements of CBC 2001 and have sufficient shear and confinement reinforcement in the upper soil layers where we anticipate lateral deformations. The attached calculations "Supplement No. 54" demonstrate that the shear strength of the pile exceeds the maximum shear from nonlinear analysis by a factor greater than 2 and is more than adequate to develop plastic hinging in the piles.	5/15/2019: Resolved
55	1/30/2019	SV	1 & 3		Pile head rotation shown on Figure C-11 of SLATE report is different from that shown on Figure 5-18 of SGH Volume 3 calculation report as wells as those estimated by LERA's team. It is noted that various mat rotation has been reported (please see attached file for comparison of reported values). If we consider ENGO's results (Analysis Case 2) with almost 1% maximum rotation in the NS direction, it would be logical to assume that during seismic loading condition, the maximum rotation would be much higher that capacity shown on Figure 1-27 (SGH report, Vol. 2). Furthermore, we note that there are inconsistencies in some analysis cases. For example, from Analysis Case 6, it is apparent that for developing backbone curve in lateral direction (Section 5.3 of SGH - Combination of Pushover Results to Obtain Composite Foundation Backbones) pile head rotation has been varied between +/- 1%. However, pile head rotations used in PERFORM 3D is based on Vol. 2 report. Please explain and clarify.	ROH/LH	2019_03_01	We respond to each of the several questions embedded here, separately: 1) ENGEO's results show total rotations and not plastic rotations. These include rigid body rotation, elastic rotation, and plastic rotation. Refer to our response to comment #50. 2) The pile head rotations used to generate the base shear backbone are based on results from SGH's Perform-3d model which simulates the deformed shape of the mat due to settlement using thermal loads assigned to the support springs. 3) The Perimeter Pile Upgrade design team cannot opine on the validity of analyses performed by the Engeo/LERA team as essential details related to the input and engineering assumptions of those analyses are not available. 4) We acknowledge that there is a significant degree of uncertainty in the estimates of pile head rotations. We therefore performed sensitivity analyses on the base shear backbone curve using twice the pile head rotation values from our analysis. Please refer to "Supplement No. 55". This results in a base shear backbone curve which is essentially the same as the one presented in our original calculations. The unbalanced loads due to shear developed in the piles increased twofold. The base lateral displacements obtained from seismic analysis, using these alternate backbones, remained close to our original values. It is therefore our opinion that the ability of the building structure to resist base shear is not sensitive to the amount of head rotations in the piles.	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
56	1/30/2019	SV	3		Figure 6-1, 6-13 & 614: please state whether dynamic soil pressure (dynamic increment) has been applied to the basement walls on the west, east, and north side. Furthermore, please explain how soil pressure on the deeper section of the mat at elevator pit has been accounted for.	JE	2019_03_01	A dynamic increment of soil pressure (due to inertial loading) is not applied to the basement walls on the west, east and north sides of the Tower and Podium. Instead, the kinematic interaction is accounted for in the non-linear soil springs attached to the substructure. This is conservative since the movement of the free field "soil" is restricted (i.e., the "free-end" of the soil springs is rigidly fixed in space). Inertial and kinematic interaction between soil and structures are typically out of phase; including both in the analysis would be unrealistic from an engineering perspective. Soil pressure on the deeper section of the mat at the elevator pit has not been considered in the analysis. The piles surrounding the elevator pit will inhibit both resistance and loading from soil pressures from developing.	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
57	1/30/2019	SV	3		Section 6.1.1 (Page 120) : 100% of mass of above grade podium has been considered to be active in the first mode. Note that for a cantilever beam with uniform mass and stiffness, the equivalent lumped mass for a single degree of freedom is 1/3 of total mass. Please clarify.	ROH/LH	2019_03_01	I tis correct that the first mode participating mass will be somewhat less than 100%. Chopra derived the equivalent lumped mass for an SDOF as 61.3% of the total mass. We made a conservative assumption to include 100% of the mass of the above-grade midrise when modeling it as a single degree-of-freedom oscillator. The higher mass in the first mode would account to some extent for base shears due to higher modes which are not modeled.	5/15/2019 - Resolved pending submission/review of the revised/final building analysis results (comment 33). 6/26/2019 - Resolved
58	1/30/2019	SV	3		Section 6.1.2 : Soil pressure acting on the south side of the tower basement wall has been neglected. Considering that the adjacent TTC train box "racks" three inches from bottom of train box to the at-grade slab during the upper level event, please justify ignoring the dynamic earth pressure transmitted to the basement wall of the tower on the south side.	ROH	2019_03_01	Supplement 58 is a cross section through the site, extracted from the Arup analysis report for the TTC shoring wall, showing the predicted displacements of the shoring wall, neglecting the presence of the terminal. Note that this figure is diagrammatically incorrect, as the ground floor slab for the tower is shown inordinately thick, while the extension of the Millennium mat beneath PG&E vault (3ft thick) is shown too thin. Under interaction between the tower and TTC, nearly all force transfer will occur through the W21 soldier piles in the TTC shoring wall. These will experience a hard point at the level of the Millennium Tower mat and most force transfer will occur through this interface. The tower's south basement wall will see relatively little force and to the extent that force transfer does occur above the mat, this will mostly be taken by the shoring wall soldier piles, which are significantly stiffer than the tower wall, and will span between the mat and first floor slab.	5/5/2019 - Resolved
59	1/30/2019	GD	8		Pile notes on Drwg. T002 indicate a maximum tolerance of pile plumb of 2%, which translates to 5.6 ft for the anticipated 280 ft pile depth. Is realistic given that the clear distance between piles is about 2.5 ft?	SKH	2019_03_01	Following discussion with the contractor, we revised the out-of-plumb tolerance to 1% and also added the following requirement to the specifications: "Track and record the as-drilled variation from vertical and tip location of each pile. If the tip location of any pile is closer to the specified location of the subsequent adjacent pile than indicated on the drawings, report the location to the Engineer. Adjust the plan location and angle from vertical of the subsequent pile to accommodate, as directed by the Engineer." We have confirmed with the Contractor that this is achievable.	5/15/2019 - Resolved
60	1/30/2019	GD	7,8	High	Drwg. S501 specifies the requirement for CP pile casing welds over the top 30 ft depth of pile. Please confirm if this is meant to be measured from grade or the top of mat and confirm that the specified depth is consistent with possible hinging in piles under large earthquakes. In addition, shouldn't these welds be specified as demand critical with notch tough filler metal (related to notes on Drwg. T002).	ROH/SKH	2019_03_01 2019_06_07	~~This is intended to be the top 30 feet below the mat extension which is consistent with the depth where hinging can be anticipated in an earthquake. We will clarify the distance on our sheets. We will indicate as Demand Critical welds~~ The results from our latest analysis show a maximum lateral displacement of 3.07 inches which is less than the anticipated yield lateral displacement of the proposed pile equal to 3.8 inches. We plan to revise S501 and T002 to show PJP (-1/8") welds that are not designated as Demand Critical.	8/5/2019 - Resolved
	6/26/2019				Please confirm whether the weld splice near the base of the mat extension (1/S502) is a complete penetration weld and whether this is demand critical (given the location of high flexural demands, especially considering flexibility of sonotube liner). Also, please confirm where (if anywhere) the welds are intended to be specified as demand critical (per the specification of CVN toughness requirements for demand critical on S001.)		2019_07_19	The welded pile splice near the base of the mat extension is a partial joint penetration weld with a thickness equal to the thickness of the pile casing minus 1/8 inch. We do not expect the pile to yield under the maximum displacements predicted by our sensitivity analyses. The expected yield flexural strength of the pile is 35,000 kip-in and our analysis predict a maximum response of 29,150 kip-in which occurs at the piles along Mission Street when the building moves to the south toward the Transbay Terminal. We no longer specify demand critical welds in the project.	

Comments by EDRT						Responses by Design Team			Date
61	1/30/2019	GD	7, 4	High	Drwg. S403 (details 2 and 3), S202 and Cals V4, 3.1 - Please confirm how the vertical headed bar shear reinforcement in the "high strength" and "regular" details is determined and the loading condition that necessitates the high strength detail in certain locations. It would be helpful to provide a sketch of a strut-tie mechanism to show how the tension force in the threaded anchor rods is transmitted back to the mat.	ROH/LH	2019_03_06	The vertical headed bar reinforcement was designed to resist the maximum of the SAFE strip shear demands and the 800-kip pile design load increased by a factor of 1.5. We updated the calculations in Volume 4, Section 3.1 to show both the regular and high strength detail checks starting on page 30. The intended shear transfer mechanism is through the spaces between the piles where these spaces are treated as beams loaded in one-way shear. In the perpendicular direction shear is resisted by concrete alone. See Supplement No. 61	5/15/2019 - Resolved
62	1/30/2019	GD	7		Drwg. S401 and S402 - Please update the drawing to show the as-built configuration of the vertical headed reinforcement in the existing mat (i.e., with the bar heads above the lower layer of mat reinforcement)		2019_06_10	The sections have been updated. Please refer to revised drawings.	6/26/2019 - Resolved
63	1/30/2019	GD	7	High	Drwg. S401 - Please confirm that the 12" embedment length is sufficient to develop the #10 bars.	ROH/LH	2019_03_01	We revised our shear friction calculations. Shear friction strength is now based on the equation proposed by Mattock in 1975 which is referenced in the commentary to ACI 318-14 Section 22.9.1.2. For the majority of the mat extension, no epoxied reinforcement is needed. At the west mat extension, approximately between Gridlines D and E, additional #8 bars spaced at 16 in horizontally and 24 in vertically are needed to meet the minimum reinforcement ratio correlated to Mattock's test data. The revised design requires 21-inch embedment length for these bars. We will update Drawings S202, S401, and S402 with the revised reinforcement. See attachment "Supplement No. 63" for design calculations.	5/15/2019: Resolved pending review of the revised drawings. 8/26/2019 - Resolved
	6/26/2016				(1) The Mattock equation (7) referenced in the Volume 4 calculations (pg. 55 and related calcs) is based on results for monolithically cast concrete (implying mu = 1.4) , whereas the mat extension interface is a roughened cold joint (implying mu = 1.0). Therefore, it seems unconservative to utilize the implied concrete shear strength of 400 psi across the joint between the mat and the mat extension.. Please review and revise the shear strength equations (Volume 4, around pg. 56) and, if necessary, provide additional steel across the shear plane at critical locations. (2) We would recommend the addition of epoxy embedded steel reinforcement around the north-west corner of the mat extension, given the high concentration of new piles and the discontinuity of the mat extension at the PG&E vault. (3) In Vol 4 (pp 56 and following) the 'max' function does not appear to be working. For example, for design strip 1 v.u is noted as 600k/ft, but the equation reads that is should be the max(266,282). Only in design strip 5 does the calculation actually show the maximum. Please review and revise. (4) The calculations of the mat extension (Volume 4, pages 19 through 21) appear to only account for flexural and shear loading at the mat interface based on jacking and overturning. There is an additional in plane shear and/or normal force (direct and from eccentricity) generated by the lateral displacement that is not accounted for. Please review and revise the calculations as necessary. (5) Please confirm the purpose of the diagonal hooked bars in Details 1 and 2 of S401 and where these are referenced in the calculations.		2019_07_19	See Supplement No. 63 for calculations addressing the comments as follows: (1) We revised the shear friction calculations to implement a shear friction equation proposed by Harries et. al. in a 2012 ACI Structural Journal publication, which we have included in our transmittal. The Harries equation is based on a dataset of new and historical testing, including tests of monolithic, cold-joint, and pre-cracked samples. Our original design was confirmed with this method. (2) We evaluated shear and moment transfer at the mat extension corner utilizing the flexural bars coupled to the existing mat reinforcement and the new mat extension shear and flexural bars. We found that an adequate load path exists for the retrofit piles at the corner of the mat extension. (3) We revised the shear friction calculations to correct the noted error in determining design strip demands. (4) We revised shear friction and flexural capacity calculations to account for additional in-plane flexure and normal loads due to lateral displacement of the mat. (5) The hooked diagonal bars are for shear friction resistance under load combinations resulting in tension at the top of the mat extension interface. We have included shear capacity calculations for this load condition.	
	8/5/2019		20	High	As shown in the figure on PDF pg. 56 of Supplement 63, the load path from pile 30 into the existing mat is indeterminate. The moment capacity calculation that you provided on the "critical section" (on pg. 56) is one possible load path, but there are alternate load paths that are stiffer and should be considered e.g., shear/torsion through the shear friction plane of Pile 29. To help mobilize all available force transfer in the NW corner of the mat extension, it is suggested to (1) confirm that the flexural steel in the Fremont street mat extension is anchored with a hook or head just below column line A, Drwg. S202, (2) add additional vertical headed bars (or other shear/flexural reinforcement) into the corner region of the mat extension to mobilize all available shear and torsion capacity in the mat extension, (3) consider using epoxy dowels into the existing mat.	ROH/LH	2019_08_06	We evaluated the transfer of shear force from Pile 30 through the mat extension interface at the northwest corner. We checked the capacities of parallel load paths at the interface tributary to Piles 29, 31, and 32. We found these load paths to be adequate to transfer all jacking shear loads from Pile 30 into the existing mat. The shear strength of the mat extension and its interface with the existing mat is therefore adequate to mobilize the available flexural strength for transfer of load in this corner. Please refer to Supplement No. 63 for calculations. We revised sheets S202, S203, and S403 to show the following details recommended by the EDRT: (1) Hooked development of Fremont St. flexural reinforcement (2) Additional vertical headed shear reinforcement in the northwest corner of the mat extension (3) Epoxy-doweled reinforcing bars in the northwest corner of the existing mat for supplemental shear friction force transfer	
64	1/30/2019	GD	7		Drwg. S502 - Please confirm if the pile loading information is up to date (during our last meeting, it was mentioned that the piles may be re-loaded twice rather than once). We further suggest that the pile loading instructions on Drwg. S502 include (1) requirements to record the date/time when each pile is loaded (or reloaded) and, if possible, a record of the relaxed pile load prior to reloading, and (2) a minimum number of days between pouring of the mat slab extension concrete and commencement of pile loading (to ensure concrete has sufficient strength).	ROH/LH	2019_06_10	The sections have been updated. Please refer to revised drawings.	6/26/2019 - Resolved
65	1/30/2019	GD	7		Drwg. S502 - Please confirm that the waterproofing detail around the steel pile can accommodate the movement of the pile expected during the preloading operation.	ROH/SKH	2019_03_01	Confirmed. This detail was intended to accommodate the anticipated preload displacement.	5/15/2019 - Resolved
66	1/30/2019	GD	7, 4	High	Drwg. S502, Detail 1 and Calc V4, Sec. 1.2 - In the calcs please (1) check whether the effective area of the threadrod is included in checking the threadrod tension yield strength (it does not appear to be considered), and (2) confirm the basis of assuming a plastic straining capacity of 5% over the full threadbar length (given the tendency for localization of inelastic strains). In Drwg. S502, please specify the intended material for the 2-1/4 inch threadrods (to avoid confusion with reinforcing steel threadbars).	ROH/SKH	2019_03_01	1) Confirmed. The 2-1/4" rod was incorrectly sized on the base of Ag rather than An. We will revise the rod size to 2-1/2 Inc. The net area of this rod is approximately the same as the gross area of the smaller rod originally specified. 2) Please see Supplement 66 which shows a stress strain curve for the material specified. It has ultimate tensile strain well in excess of 5%.	8/5/2019 : Resolved
	5/15/2019				Tension testing per ASTM E8 (as referenced in Supplement 66) is based on standard machined round test coupons, results of which are not indicative of the tension load-deformation response of the threaded bars. As discussed, test data of the threaded bars is needed to confirm that they can provided the assumed displacement ductility in the pile fuse detail.	SKH	2019_06_07	We have performed full-scale testing of the F1554 Grade 55 threaded rods and have verified that they can sustain strains well in excess of the anticipated 5% strain prior to failure. The tests showed elongation exceeding 21%. A full report is pending	
	6/26/2019				Resolved pending clarification of a couple things from the rod test report: (1) based on the ratio of reported bar yield stress (58 ksi) to yield strain (0.38%), the calculated elastic modulus is about 15,300 ksi, which is much lower than expected. Please review and confirm. (2) please confirm the bar area used to convert applied force to stress (3) please confirm how the strains in the plot on page 3 were determined. The report implies that the strains were determined from the extensometer readings, but since it appears that bar necking occurred outside of the 2" extensometer range, it seems that the strain (at least the large strain portion) is based on crosshead displacement. Please confirm.		2019_07_19	1) We reported the strain at 0.2% offset where the yield stress was determined. This strain is therefore not appropriate for use in elastic modulus calculations. The elastic modulus of the steel as tested is approximately 29,000 ksi as would be expected. 2) We computed the net area of the bars based upon measurement of the threads as 4.0 square inches which is consistent with the value listed for a 2-1/2-inch diameter rod in AISC 360-16 page 7-83 Table 7-17. 3) Two extensometers were used in the test - a 2-inch and a 9-inch gauge length. Strains were reported from the 9-inch-long gauge where necking and crosshead displacements did not occur.	

Comments by EDRT						Responses by Design Team			Date
67	1/30/2019	GD	1		Table 3-1: Please include designations for moment frame columns and beams in the table. In addition, for the foundation piles, please specify how shear of the structural component of the piles is considered (e.g., shear, force-controlled, critical?). It would also be helpful to differentiate criteria between existing precast RC and the new composite pipe piles.	ROH/LH	2019_03_01	We added the requested items to Table 3-1. Please refer to the updated Revision 2 - Volume 1 Design Overview.	5/15/2019 - Resolved
68	1/30/2019	GD	2	High	Calcs V2, pg. 5 indicates that the analysis and design considers a hydrostatic pressure based on a watertable that is 3 ft below grade. Being as the observed water table is much lower (28 ft is mentioned in the shoring report), please confirm there are no instances where the assumed height leads to an unconservative design check (i.e., where the hydrostatic pressure relieves gravity-induced or earthquake-induced effects on the mat, piles, or other elements).	ROH/LH	2019_03_04	The effect of the location of the water table on mat stress is small, as the hydrostatic pressures are applied as uniformly distributed point loads, which are transferred by the mat to the nearest piles, with negligible shear or bending influence on the mat. For purposes of lateral evaluation of pile capacity, we used the axial pile loads recommended by Egan, based on the FLAC analyses and these were computed assuming a water table depth of 10 feet below grade, which is the historic level (prior to dewatering)	5/15/2019: Resolved pending review of the final review of the foundation bounding analyses, as discussed on 4/23/2019 6/26/2019 - Resolved
69	1/30/2019	GD	1, 3		CalcsV1, Table 3-2 and Calcs V2 Table 1-2 and Fig. 1-8: The assumed expected strengths of the 10,000 psi concrete (13,000 psi, unconfined and 19,000 psi confined) are quite high. PEER TBI V2 cautions that the expected strength multiplier of 1.3 may be smaller for high strength concrete, and AISC TC 5 recently proposed to change the multiplier to 1.2 for f'c > 7ksi based on a study by Nowak and Szerszen (Calibration of design code for buildings: Part 1, 2003). Two additional questions are (1) whether Mander's confined concrete strength parameters apply for high strength concrete, and (2) whether there should be a reduction factor to relate concrete cylinder strengths to effective strengths in concrete members (C factor in Section 4.4 of Moehle's 2015 textbook). Please confirm the basis for the assumed values, or alternatively, confirm whether the analysis results and resulting design checks are sensitive to these assumed values.	ROH/LH	2019_03_04	This comment only effects the adequacy of the core walls to resist earthquake demands. The proposed voluntary seismic upgrade does nothing to change these demands. Regardless, as demonstrated by our lateral analyses of the structure (e.g. See Vol 3, Figures 6-27 and 6-28) compressive demands on the concrete walls are very small, with maximum predicted compressive strains much less than 0.003. Therefore, whether or not Mander's confinement model is completely valid for the high strength concrete, we are not in a portion of the material response where this matters.	5/15/2019 - Resolved
70	1/30/2019	GD	2	High	Calcs V2, Fig. 1-26 and pg. 38 (and related to comment #25): Please confirm that the calculated mat shear strengths are consistent with the as-built vertical shear reinforcement in the mat, which is anchored above the lower plane of reinforcing steel. Also, in Figure 1-26, please confirm that the reported values are expected strengths (as opposed to nominal or design strengths) for the 5-ft width of the grillage beams.	ROH/LH	2019_1_30	Please refer to our updated calculation in "Supplement No. 25". The reported values are expected, as indicated.	8/5/2019 : resolved
	5/15/2019				As discussed during the meeting of 4/23/2019, please provide checks to confirm that the shear capacity exceeds the imposed shear demands over portions of the mat (4 segments along the NS direction, and 3 segments along the EW direction)		2019_06_07	Our revised analysis package (provided to the EDRT on 5/20/2019) includes checks for mat shear demand over the requested subsections.	
71	1/30/2019	GD	2		Calcs V2, Fig. 1-24 and 1-25: Please confirm the definition/meaning of the title "about N-S" and "about E-W" axis. Does it refer to grillage beams running in the N-S and E-W directions? Or, does the term axis refer to the bending axis of the members, which would be in the direction normal to the members. Also, please (1) confirm that the reported capacities in Fig. 1-24, 25 correspond to expected member strengths (versus nominal or design strengths), and (2) the amount of flexural steel used in the calculations on pg. 38.	ROH/LH	2019_1_30	In Figures 1-24 and 1-25, "about the N-S axis" refer to the bending axis of the members. Therefore, Figure 1-24 indicates flexural capacities of grillage elements oriented in the E-W direction. We confirm: (1) We calculated the reported member strengths for expected material properties (fc' = 9,100 psi and fy = 82 ksi) (2) The strength calculations on page 38 (Volume 2) base the moment capacity "Mbp" from results of cross section analysis using computer program spColumn. The analyzed section is one foot wide, with two #11 bars top reinforcement and two #11 bars bottom reinforcement. The capacity corresponds to the region labeled "1-H1" in Figure 1-24. See attachment "Supplement No. 71" for revised Figures 1-24 and 1-25 and example calculations.	5/15/2019 - Resolved
72	1/30/2019	GD	2		Calcs V2, Figure 1-29: Please provide or direct us to information to identify where the pile zone regions 1 to 6 are defined in plan.	ROH/LH	2019_1_30	Refer to Figure 1 in Supplement 72 for a plan of pile property zones.	6/26/2019 - Resolved
	5/15/2019				Thank you for providing this figure. It would be useful to include this in the final set of calculations. More importantly, please confirm whether the ENGEO pile stiffness and capacities are being used as the basis for the final analyses. If they are, the EDRT either needs access to the ENGEO values or to supporting calculations by SLATE.		2019_06_07	Our revised calculations do not rely on ENGEO recommendations for any geotechnical parameters. Please refer to the updated calculation volumes provided to you on 5/20/2019.	
73	1/30/2019	GD	2		Calcs V2, Figure 1-31, 32 and 33: Please confirm how to interpret the mat rotation values shown in the figure. Many of the grillage beams show a numerical value of "0" and others (in green) show values of 0.01 to 0.02. Are these values of radians? How should they be interpreted relative to the mat hinge acceptance criteria of 0.01 rad in V1?	ROH/LH	2019_1_30	Figures 31-33 show DCRs for the mat rotation relative to the defined capacity of 1% plastic rotation. To get the rotation in the mat flexural hinges, multiply the DCR shown in the figure by the capacity (1% rotation). Therefore, a DCR of 0.01 plotted on the figure equals to a plastic rotation of 0.01*1% = 0.01%.	5/15/2019 - Resolved
74	1/30/2019	GD	2		Calcs V2, pg. 52. Please complete the sentence, "The depressions were not modeled in the…"	ROH/LH	2019_1_30	The sentence should read: "The depressions were not modeled in the SAFE analysis." The volume has been corrected.	5/15/2019 - Resolved
75	1/30/2019	GD	2		Calcs V2, Figure 2-26: The figure caption indicates that top and bottom steel areas are shown, but most of the mat area only shows one steel area. Is this number correct? And, if so, does it correspond to both NS and EW?	ROH/LH	2019_1_30	Figure 2-26 shows steel areas for bars running in the north-south direction only. Figure 2-27 shows steel areas for bars running in the east-west direction (previously the figure caption was mis-labeled). For clarity, we have revised the figures to explicitly call out top and bottom rebar areas, including regions where top and bottom steel areas are the same. For consistency with our analyses, we have also revised the figures so that they are divided based on the design strips we used in our design analyses. See the attachment "Supplement No. 75" for revised Figures 2-26 and 2-27.	5/15/2019 - Resolved
76	1/30/2019	GD	2	high	Calcs V2, Fig. 2-34 and 2-35: - Please confirm how the capacities are calculated. Is this a standard building code check, where design strengths are used? Or are you using the expected strengths? - Please clarify why the reported DCR values are less than 0.9, but the plot shows some orange and red locations, which according to the legend are > 0.9. Please provide a short written narrative to summarize and explain the implications of the mat DCR values, including (1) sensitivity of results the foundation spring models (ENGEO versus SAGE), and (2) effect of the foundation retrofit as related to building code requirements and performance in general.	ROH/LH	2019_03_01	1) We used nominal strength and reduction factors in accordance with ACI 318-14 to compute capacities. Please refer to Figure 2-23. 2) The plots show color coded flexural DCR checks at 2-foot intervals along each strip. In areas where the individual strip DCR is greater than 1.0 we added demands and capacities at multiple strips to compute average DCRs. This is represented in our plots by arrows perpendicular to the strip orientation. 3) The ENGEO springs are somewhat softer than the Egan springs and result in higher load demands in the mat. However, we only used the ENGEO properties to study response sensitivity and to compare our results with those from LERA. 4) The foundation retrofit results in improved lateral strength at the building base and a different moment distribution in the mat. We demonstrated in our calculations that the retrofitted foundation complies with the requirements of the building code. Performance remains essentially the same except lateral displacements at the base where we see an improvement due to the addition of the new piles. Please also see our response to comment 77 and "Supplement No. 77" summarizes performance of the retrofitted foundation.	5/15/2019: Approach okay, except that the mat shear should be checked in segments (4 segments NS and 3 segments EW). Pending review and approval of the updated analyses with the bounding foundation properties and agreed upon ground motions. 6/26/2019 - Resolved
77	1/30/2019	GD	3	high	Calcs V3, Section 6. The current document evaluates the performance of the building with the existing foundation condition, assuming that this is conservative relative to the retrofitted foundation. Given that the foundation retrofit is stiffener and is non-symmetric, further analyses (or other supporting evidence) is needed to confirm the performance of the building with the retrofitted foundation.	ROH/LH	2019_03_01	We modeled the foundation retrofit in PERFORM and analyzed the tower using the suite of MCE ground motions. We found that performance is acceptable in the retrofitted condition. The attachment "Supplement No. 77" summarizes how we modeled the foundation retrofit and shows the corresponding analysis results. Comment 2 addresses the nonsymmetrical condition.	5/15/2019: See comment 76. 6/26/2019 - Resolved
78	1/30/2019	GD	3		Calcs V3, Section 5. Please confirm the assumed length of the pile hinge to convert between XTRACT cross-section curvature response and SAP hinge rotation response.	ROH/LH	2019_03_01	We assumed a plastic hinge length of 14 in., equal to one pile least cross section dimension which is conservative relative to criteria provided in CBC 2016-Section 3107F.2.5.3 . Please refer to our response to Comment 122 for further details on plastic hinge length.	5/15/2019: Resolved. It would be helpful to incorporate this into the final calculations. 6/26/2019 - Resolved
							2019_06_07	Please refer to the updated Calculation Volume 3, Section 5.1 - revision 5.	
79	1/30/2019	GD	3		Calcs V3, Section 5.4. To further establish that the detailed pile group analyses are picking up overturning effects, please compare plots of (1) axial pile force versus time, and (2) pile shear versus lateral displacement for the pair of end piles in the analysis with and without overturning effects. The point is to confirm whether the model is picking up the overturning effects on individual pile response.	ROH/LH	2019_03_01	The attachment "Supplement No. 79" contains the requested plots. The results confirm the model picks up the intended +/- 300 kip axial force overturning effects.	5/15/2019 - Resolved
80	1/30/2019	GD	4	high	Calcs V4, Section 3.1: Flexural Check of Mat Extension - When checking the bottom reinforcement moment strength of the mat extension, does the analysis consider the moment demand due to the local bending of the pile (e.g., similar to how you've considered local moment in evaluating the top reinforcement in Section 3.2)? - Please confirm that the overall design checks on page 26 are based on the design strength of the mat (i.e., using phi-factors) and the justification for averaging the values of Vu_SAFE,1 and Vu_SAFE,2. - Please confirm the factor of safety for the mat extension under the ultimate pile force (based on thread bar yielding) and the expected strength of the mat extension.	ROH/LH	2019_03_01	1) We confirm we considered local pile bending in the mat extension design. 2) The calculations on page 26 are not a design check but rather a verification of the SAFE model demands. Please see the attachment "Supplement No. 80" for explanation of the calculations. 3) We computed the mat extension capacities using nominal material properties with phi-factors. Comparing to moment demands resulting from 1200 kips per pile, (safety factor of 1.5 on the pile axial force), we obtained DCRs ranging from 0.65 to 0.8. Expected yield strength of the thread bar, including strain hardening is 1150 kips. Thus, the minimum factor of safety is about 2.0.	5/15/2019 - Resolved

Comments by EDRT						Responses by Design Team			Date
81	1/30/2019	CS	5		Section 7.1 Current groundwater predicted to be 23 feet based on drilling performed in October 2017. No rise in groundwater in 1-1/2 years?	JE	2019_03_06	Piezometers installed around the Tower have been recording pore pressure prior to and following the groundwater level observed during drilling performed in October 2017. The piezometer readings have suggested recovery between 0 and 2 feet in that time frame. Text will be revised to read: "In October 2017, the Millennium Tower Homeowner's Association (MTA) project team observed groundwater at about 23 ft bgs (elevation NAVD88-8ft) i.e. 2ft from the bottom of the 10-ft mat during the mat investigation exploration drilling performed in the Tower basement. There are currently seven arrays of piezometers installed at various times around the Tower (as shown on Figure 5) by Cotton Shires and Associates (CSA) in 2016, by Arup in 2018 and our project team in 2018. The measurements among these piezometers, although somewhat different, exhibit reasonably similar piezometric head conditions and trends, and that the groundwater has begun to rebound only slightly (about 1 to 2 feet) following the completion of the excavation activities at the properties adjacent to the Tower."	8/5/2019 : Resolved
82	1/30/2019	CS	5		Section 8.0. References ENGEO (2018) report/memo not provided to peer review team. Please provide for review.	JE	2019_03_08	The report will be revised to be self-sufficient. All references to ENGEO will be removed.	8/5/2019 : Resolved
83	1/30/2019	CS	5	high	Section 9.2. How was settlement from secondary compression analyzed using 1D analysis?	JE	2019_03_06	For each model stage, stress states were extracted from each FLAC3D model zone. 1D settlement calculations were performed for each zone, using these stress states. To calculate secondary compression, we selected zones that were at or near an OCR of 1.0, then performed a typical secondary compression settlement calculation as $S_s = H*C_{c\varepsilon}*(C_\alpha/C_c)*\log(t/t_p)$. We estimated t_p based on Terzaghi theory and the observed settlement at the site. H is the height of each FLAC3D zone.	5/15/2019 - Resolved
84	1/30/2019	CS	5	high	Section 9.2. How were loads from FLAC3D modelled in the 1D analysis?	JE	2019_03_06	See response to Comment 83; the stress states were extracted from each FLAC3D model zone, for each model state.	5/15/2019 - Resolved
85	1/30/2019	CC	5		Section 9.2. Provide graph showing Pp' from results of available consolidation tests to justify Pp' profile used in analysis	JE	2019_03_06	See response to Comment 11. Pp' data included in plot.	5/15/2019 - Resolved
86	1/30/2019	CS	5		Section 9.2.5. What current groundwater level was used in analysis?	JE	2019_03_06	The initial groundwater level was 5 feet below ground surface. This groundwater level was used throughout the FLAC3D modelling. Changes in groundwater were accounted for in the 1D settlement calculations outside the FLAC3D model by increasing the effective stress in the zones by $\Delta\sigma'v = \gamma wdd$, where γw is the unit weight of water and dd is the effective groundwater drawdown (see Table A-3). The 1D calculations assume that the current (2019) groundwater level includes the effective drawdown.	5/15/2019 - Resolved
87	1/30/2019	CS	5	high	Section 9.2.5. Report states laboratory tests on the OBC are currently being performed? Provide results of tests in report including table summarizing current Pp', OCR, Cec, Cer, and $C\alpha$	JE	2019_03_06	See response to Comment 37 and Comment 99	8/5/2019: Resolved
88	1/30/2019	CS	5		Section 9.4. Report states additional calculations are being performed. When will the results of those calculations be presented?	JE	2019_03_06	The results of these calculations will be presented when they are completed and checked	8/5/2019 : Resolved
89	1/30/2019	CS	5	high	Section 10.5. Presence of waterproofing below the mat foundation for the podium and on the podium basement walls will reduce friction values below those recommended in the report.	ROH	2019_03_06	Barrier waterproofing was not provided in the original construction. Waterproofing was achieved through use of hydrophilic admixture in the foundation and basement wall concrete.	5/15/2019 - Resolved
90	1/30/2019	CS	5		Section 13.0. Report states jet grouting will result in compressive strengths of 500-2,000 psi in soil-cement mixture based on previous testing. Does that previous testing include mixtures of cement and Bay Mud?	JE	2019_03_06	Yes, the jet grout soil-cement test results SGH has from a previous project nearby includes jet grouting through Bay Mud, as well as Colma Sand. As a note, the cited range of compressive strengths is typical of a wide range of soil types and initial conditions reported in publications by various authors. We are aware of published test results data (Ivanetich and Shao, 2008; citation below) that indicate unconfined compressive strengths less then 500 psi for soil-cement specimens of jet grouted very soft Young Bay Mud. That data also suggests that the strength increases with age, having doubled over time from an average unconfined compressive strength of about 200 psi at an age of two weeks, to about 300 psi at four weeks, to about 400 psi at eight weeks. •Ivanetich, K., and Shao, L., 2008, Jet Grouting for Mass Treatment to Support an Aggregate Stockpile over Very Soft Clay, Proceedings, Sixth International Conference on Case histories in Geotechnical Engineering, Paper 7.69a, 6p., August.	5/15/2019 - Resolved
91	1/30/2019	CS	5		Section 14.2. Report recommends monitoring of groundwater level, but monitoring plan on shoring drawings does not show any new wells or piezometers. Where will groundwater be monitored?	JE	2019_03_06	This is currently being revised by the retrofit design team subject to approval/discussion with the EDRT. We will update the text once the planning is finished. The current plan is to perform one boring through the existing foundation mat at the north end of the Tower and one outside of the retrofit trench, and then install piezometers in both borings	5/15/2019: Resolved - based on piezometer to be installed under mat (permit approved) and at proposed exterior location along Fremont street.
92	1/30/2019	CS	5		Section 14.3. Fill is likely contaminated and will need to be profiled prior to disposal per Maher ordinance	JE	2019_03_06	Agreed. Text added to Section 14.5 "Earthwork" of report as follows: "Site soil deemed contaminated will be profiled per the Maher ordinance prior to disposal. Site soil not deemed contaminated will not be used as fill material due to space limitations for tits storage during construction on the site."	8/5/2019 : Resolved
93	1/30/2019	CS	5		Section 14.3. Aeration/re-use of fill is not feasible.	JE	2019_03_06	Agreed. Even if the fill were not potentially contaminated and suitable for re-use as backfill, it would be impractical due to severe space limitations for its storage during construction on the site. See also response to Comment 92	8/5/2019 : Resolved
94	1/30/2019	CS	5		Figures 4 and 5. The soil layer description of "LOWER YOUNG BAY MUD" is not consistent with description of this layer in the report.	JE	2019_03_06	Clarifying text added in Section 7.0 of the report as follows: "A soft to medium stiff marine clay deposit, known locally as Recent Bay Deposits or Young Bay Mud (YBM), is present beneath the fill. The thickness of the YBM ranges from 20 to 30 feet and is generally thicker on the west side of the site. The YBM is generally underlain by a zone of stiff to very stiff sandy clay (labeled as "Lower Young Bay Mud" in Arup, 2010) interbedded with medium dense to dense clayey sand and sand with clay (labeled as "Upper Marine Sands" and "Lower Marine Sands" in Arup, 2010) to depths of approximately 90 to 100 feet bgs (elevation NAVD88 -75 to -85 feet). This zone generally represents a stratigraphic unit of marine deposits that was characterized in some studies (e.g., Treadwell & Rollo, 2005) based on a geotechnical perspective, and in others (e.g., Cotton, Shires & Associates, 2016; Arup, 2010) from a detailed geologic perspective."	8/5/2019 : Resolved
95	1/30/2019	CS	5	high	The soil properties in Tables A-3, B-1, C-1 and C-2 are not consistent between the 4 tables. Of most concern is "Best Estimate" recompression index (0.1) in Table A-3, which appears low. Also, the values of compression index and recompression index appear to be compression ratio and recompression ratio	JE	2019_03_06	Table A-3 does not have any similar properties as compared to Table B-1, C-1, and C-2. We will assume that the reviewer meant Table A-1 for the response. The properties in Table A-1 are intended for use in the FLAC3D model, which is primarily governed by the consolidation properties rather than the long term (drained) strength properties; therefore, while a comparison is possible with the properties in the other tables, it is irrelevant for the analysis. Additionally, the properties in Table B-1 and C-1 are for dynamic (undrained) analyses, so the values should not be the same between the two sets of tables. Values are reported for completeness. The properties in Tables B-1 and C-1, while slightly different, are not substantially different enough to change the recommendations. Table C-2 provides values from the analysis by ENGEO (2018). We do not endorse the specific values used, but rather present the values for completeness of the discussion in the text. The "best estimate" is for the variable "X" used in the equation in the third column in Table A-3. Therefore, the "best estimate" for the recompression ratio is Cre = 0.12*Cce + 0.01*X = 0.12*(0.28) + 0.01*(0.1) = 0.0346, which is 12.4% of Cce. Correct regarding index versus ratio; we will update the description in the table. See Comment 38 regarding Cre parameters.	8/5/2019: Resolved
96	1/30/2019	CS	5		Figure G-1. Assuming concrete trucks will be temporarily parked next to shoring during construction of the piles, the surcharge pressure for design of the shoring should be higher than 100 psf (typically 250 psf).	SY	2019_06_07	Shoring was designed and checked for both the traffic surcharge (100 psf lateral) and construction surcharge (250 psf). Please refer to Figure 3-9 from 2018-12-05 Shoring Design Report Permit Submittal dated 12/05/18.	8/5/2019: Resolved
97	1/30/2019	CS	5		Figure G-1. What is depth of grout plug?	JE	2019_03_06	According to Document 10, the design thickness of the grout plug at the base of the excavation is five (5) feet	5/15/19: Resolved
98	1/30/2019	CS	5		Figure G-2. It is not clear what is "displacing" or "deflecting". Is this the jet-grout columns below the excavation? Maximum lateral deflection per drawings is 1.5 inches. The jet grout compressive strength on this figure (500 psi) is higher than that called out on drawings (400 psi).	JE	2019_03_06	Figure G-2 shows load-deflection curves for the shoring wall, for soil only. SGH used these curves and the pressures in Figure G-1 to model the shoring wall. For completeness, the curves extend to 20 inches of deflection, though the allowable deflection is 1.5 inches. SGH developed their own load-deflection curve for jet grout based on a compressive strength of 400 psi. The drawings and report will be corrected to show consistent compressive strengths for jet grout. We will modify our recommendation and associated value on the figures	8/5/2019: Resolved
99	1/30/2019	CS	5		Log of SAGE Boring B-1. The results of laboratory tests performed on soil samples from this boring should be presented as a separate appendix in the report, including drained parameters from TX-ICU tests on OBC	JE	2019_03_06	See Supplement 99	8/5/2019: Resolved

Comments by EDRT						Responses by Design Team			Date
	5/15/2019				Please provide supplement 99 to EDRT (it seems to be missing from updates)	JE	2019_06_07	Re-submitted with this package.	
100	1/30/2019	CS	6		Sheet S003. Jet Grouting. At least 4 of the 6 samples of the soil improved with jet grouting should be taken in the Bay Mud (i.e., below the sandy fill) since that strength will be govern.	SY	2019_04_19	The notes were revised to include the requirements that at least 2 of 3 wet samples from each jet grout column should be taken in Bay Mud. Please refer to the attached S003. (Supplement 100)	5/15/19: Resolved
101	1/30/2019	CS	6		Sheet S003. Jet Grouting. Specifications for jet grouting are referenced. When will those be submitted for review?	SY	2019_03_04	Please refer to Specification Section 31 05 50 in the Newforma Shared Specifications folder	8/5/2019: Resolved
102	1/30/2019	CS	6		Sheet S003. Monitoring. Since the inclinometers will only extend as deep as the soldier piles, an optical survey prism should be placed on each soldier pile with an inclinometer so the deflection of the top of the soldier pile is known.	SY	2019_03_04	The top ends of all soldier piles are structurally connected to a WF beam so that the piles would deflect together. It is not practical to monitor each individual pile. Multiple optical prisms will be placed on the beam so that the deflections of the beam and the pile top can be measured. The locations of the optical prisms are shown on Sheet 212	5/15/19: Resolved
103	1/30/2019	CS	6		Sheet S003. Monitoring. Typically, monitoring includes some surface points on the street adjacent to the shoring.	SY	2019_03_04	The surface reference points will be selected by the surveying company and submitted to SGH for approval prior to monitoring instrument installation.	5/15/19: Resolved
104	1/30/2019	CS	6		Sheet S003. Jet Grouting. The notes state the contractor shall remove the excavation shoring within 10 feet of finish grade elevation. It is not clear how that can be accomplished without installing additional shoring. Suggest removing to depth of 4 or 5 feet so sloping excavation can be used.	SY	2019_06_07	We are required to remove shoring within 10 feet of the finish grade elevation. Note that the excavation backfill sequence has been revised. After the perimeter piles and vault are installed, the soldier piles will be braced at the reinforced concrete vault slab. The soldier piles will then be cut, and the excavation will be backfilled. The last step is the removal of the top whaler and struts, followed by extraction of the top 10 ft of the soldier piles. Please refer to revised drawings	8/5/2019: Resolved
105	1/30/2019	CS	6		When will calculations for the shoring be submitted?	SY	2019_03_04	The calculations report for the shoring design has been submitted. Please refer to the volume "Shoring Design Report Permit" in the Shared Calculations folder on Newforma.	8/5/2019: Resolved
106	1/30/2019	CS	8		Sheet T002. Pile Testing Notes. Note 6 calls out for removing test pile to 10 feet below grade, which we assume will be done after the shoring is installed; however, since the pile will interfere with the mat extension it will need to be removed to below the bottom of the extension.	ROH/SKH	2019_1_30	This is correct. We will revise the note to indicate removal to 1 foot below the bottom of the excavation. This will preclude the test pile from talking any load as the mat extension continues to settle.	5/15/2019 : Resolve pending review of the updated drawing notes. 8/5/2019 Resolved
107	1/30/2019	CS	8		Sheet T002. Locations of strain gauges to be installed in the test pile should be shown on Detail 1. As noted above, it is recommended that 3rd O-Cell be installed a few feet below the top of the OBC to determine how much load is being transmitted by the skin friction on the CLSM.	ROH/SKH	2019_1_30	We will show the strain gauges on the test pile elevation. The total number of gauges will be increased, in order to understand the amount of load resisted in the upper portions of the pile. Strain gauge locations will be as follows: 1) bottom of outer casing (top of OBC), 2) bottom of OBC, 3) top of Franciscan formation, 4) 10 ft below top of Franciscan formation (approximate bottom of weathered material), 5) midway between two O-cells, and 6) pile tip. We do not plan on incorporating an additional O-cell.	8/5/2019 : Unresolved, pending resolution to comment 16
108	1/30/2019	CS	7		What is specified minimum strength of CLSM? Is there a maximum strength to reduce load transfer to the soil around the CLSM?	ROH/SKH	2019_1_30	We have specified the CLSM to have a maximum undrained shear strength of 500 psf – approximately the same strength as the Bay Mud. We have indicated that the CLSM will be made from a cement-bentonite-water mixture.	5/15/19: Resolved
109	1/30/2019	MIS	2		Please discuss the difference between the modal response presented in Table 1-1 of Volume 2 and Table 2-3 of Volume 3	ROH/LH	2019_03_04	The ETABs model use cracked section modifiers for the wall piers and link beams. In the Perform model, we used cracked section modifiers for the link beams, but the wall piers are modeled as fiber elements, and behave as uncracked during modal analysis. This is common in analysis and design of high rise structures.	5/15/19: Resolved
110	1/30/2019	MIS	2		Please clarify the difference between reinforcing steel compressive and tensile behavior as noted on page 10. Figure 1-9 does not present different curves for tension and compression.	ROH/LH	2019_03_04	The language suggesting that different compressive and tensile models were used for reinforcing steel is a holdover from initial analyses, and was done to address potential bar buckling. In our most recent analyses, we account for the low strain demands in the reinforcing, and the low likelihood of buckling, and so use the same force-deformation behavior in compression and tension.	5/15/19: Resolved
111	1/30/2019	MIS	2		Please clarify how the eccentricity between the floor slab and coupling beams is addressed. Section 1.2.2 indicates only one wall element is used over the story height, suggesting that the centerline of the floor and the coupling beams are modeled at the same elevation where in reality some eccentricity occurs.	ROH/LH	2019_03_04	The model adopts the industry standard convention for buildings of this type, of assuming that the floor slabs are at the same elevation as the centroid of the coupling beam. We believe this introduces small error and regardless is consistent between the "as-built" and "retrofit" models so will not affect the evaluation of the effect of the retrofit in not reducing the building's performance.	5/15/19: Resolved
112	1/30/2019	MIS	2		Please provide a copy of the Canbolat report referenced in Section 1.2.2.3 used for the outrigger beam modeling.	ROH/LH	2019_03_06	Refer to Supplement No. 112 Appendix A for the Technical Paper by Canbolat et al.	5/15/19: Resolved
113	1/30/2019	MIS	2		Section 1.2.3 suggests that some of the moment frame columns are shear critical. Please discuss how this affects the performance of the frame, and how retrofit might improve overall performance of the building.	ROH/LH	2019_04_19	Some of the columns have shear capacities less than the shear demand at flexural yielding, taken as 2*Mp / L. The columns were not originally designed for this level of shear because the shear walls were presumed to prevent high shear demands in the perimeter columns. We explicitly evaluated the columns for MCE demands and the force-controlled provisions from TBI. We found that performance of the columns is acceptable in the retrofit condition. Please refer to the attached "Supplement No. 113" for the results of our analyses.	5/15/19: Resolved
114	1/30/2019	MIS	2	High	Please clarify section 1.4.1. The last sentence states that "The mat performance is adequate as it does not form a hinge across the gross section." However, the paragraph references a 1% rotation, which exceeds the yield rotation calculated on page 38 (6.45x10-5).	ROH/LH	2019_03_06	We refer to "a hinge across the gross section" as the formation of a mechanism in the mat (i.e., yielding all grillage elements across the full width of the mat at the same point in time). Local yielding in multiple grillage elements is acceptable and would not constitute a complete hinge or mechanism.	5/15/19: Resolved
115	1/30/2019	MIS	2	High	Section 1.4.2, please report pile forces for D+L loads before and after implementation of the retrofit from the P3D model. Also, compare P3D pile forces with forces calculated by the SAFE model presented in later paragraphs. How do these pile forces compare to what is calculated by SAGE?	ROH/LH	2019_03_06	See the figures in Supplement 115 for the pile loading.	5/15/2019: Resolved pending review of results from foundation spring bounding analyses. 6/26/2019 - Resolved
116	1/30/2019	MIS	2	High	Section 2.3, please discuss/clarify the rational for the design strips used to verify the mat performance in SAFE. More traditionally design strips are assigned based on column locations, identifying column and middle strips. How would using narrower strips affect the results? (applies also to mat verification in Volume 3)	ROH/LH	2019_03_06	Design strip widths are based on the guidelines in NEHRP Seismic Design Technical Brief No. 7 Section 8. See Supplement No. 116	08/26/2019 - Resolved
	5/15/2019				The comment response does not answer the question directly (i.e., are the results sensitive to the strip width). The provided reference is useful, but does not fully justify the proposed strip widths, as it simply discusses some general considerations and highlights some area of disagreement in code development. Also, this is a verification of an existing mat rather than a new design, which will require some sensitivity of the strip layout to the existing distribution of reinforcing, rather than a somewhat arbitrary strip width. Is the layout of design strips from the original design known? This may be a better gage to understanding if certain mat areas are seeing higher loads due to the retrofit design. If it is not known, please provide an alternate strip layout that more closely follows the column bays, and report if this results in significantly less favorable D/C ratios.		2019_06_07	See the attached calculations for mat shear results considering narrower strips. DCRs in nearly all strips are well below 1.0. A few isolated strips have somewhat higher DCRs. Refer to Supplement 116.	
	6/26/2019				Please justify the DCR acceptance criteria. The reported values of DCR up to 2.28 (pages 8 and 10 of supplement 116) are larger than what one would consider "somewhat higher" than 1.0. Related to this, please (1) provide a breakdown of the shear loads (D, L, E and J) and the design strengths for each strip, and (2) how the retrofit will increase the shear and flexural demands in the most critical portions of the mat (i.e. provide a side-by-side comparison of demands with and without the retrofit).. Also, (4) confirm whether the table of DCRs on pg. 8 is for the 'north' strips, since the headings in the left column refer to "West' strips, and (5) that the reference to "new east" in the figure on page 3 is a typo (seem it should be 'new West')		2019_07_19	The analysis using 10 ft strips was performed to understand the stress distribution in the mat foundation before and after the retrofit. The strips have a width equal to the mat depth and it is not likely that failure of the mat will occur over a single strip width. The calculations in Section 7.7 of Volume 3 show DCRs for more appropriate 25 ft strip widths, which are consistent with common design practice and with the recommendations of NEHRP Seismic Design Technical Brief No. 7. 1.The attached calculation "Supplement 116" includes breakdown of the loads and design strengths of each strip. . 2.The attached calculation "Supplement 116" includes a comparison of the factored shear and flexural demands and design strengths of each strip. 4.The DCRs on pg 7 of the attached "Supplement 116" are for the north strips. We corrected the labels in these summary tables. 5.We renamed the design strip in the figure on page 3 of the attached calculation "Supplement 116" below to "New West" and updated the figure below.	

Comments by EDRT						Responses by Design Team			Date
				high	Please confirm (1) that the calculated shears at the edge of the mat are consistent with the assumed load transfer into the new piles and (2) that the existing mat and the transfer into the mat extension can resist the induced shears. [Note - we assume that the third column on pg. 17 of supplement 16 should be labeled as "core south" rather than "core west"].		2019_08_09	1.Our prior calculations reported shears at the edge of the mat from a SAFE model which includes the perimeter basement walls and ground level slab. Additionally, it includes the mat extension which is present in all load cases including not only jacking but also dead and live loads. We studied load transfer from the new and existing piles into the strips of the mat extension and found that the perimeter basement walls (modeled linear elastic) acted as a composite cross section with the ground level slab, the existing mat, and the mat extension. These walls caused a major portion of the mat edge shears reported in our previous calculations. We reviewed the vertical forces exerted by these walls on each of the mat strips and found that some of those forces exceeded the tensile capacity of the vertical bars in the wall in these areas. We then recalibrated a stiffness modifier of 0.2 on the perimeter basement walls to limit the amount of this force to the yield capacity of the wall. We believe this results in more realistic estimates of the mat strip shears along the mat perimeter. In addition to modifying the stiffness of the walls we also removed the mat extension from the model which we use to report gravity load stresses prior to jacking, since these forces will not be affected by the presence of the mat. This resulted in a redistribution of shears along the edge of the existing mat. Finally, the edge of mat locations where we previously reported higher shear stresses were within a distance d from the tower columns. We are now reporting shear demand at a location d away from the column as permitted by ACI 7.4.3.2 and ACI 9.4.3.2. See page 9 of this document for further information on this change. 2.As previously discussed, the analysis using 10 ft strips was performed to better understand the stress distribution in the mat foundation before and after the retrofit. The adequacy of the mat is demonstrated in calculations shown in Volume 3, Section 7.7 where 25 ft strip widths, consistent with common design practice and with the recommendations of NEHRP Seismic Design Technical Brief No. 7 are used. For EDRT's reference we updated the DCRs for the 10-foot strips with columns within d of the edge of mat as described above. The existing mat is checked fo shear forces d away from the column toward the interior of the mat. The reduced demands at this location result in DCRs less than 1.0 for almost all strips of the existing mat and DCRs modestly exceeding 1.0 in a few strips. We previously checked the forces in the mat extension (see Volume 4	
117	1/30/2019	MIS	3	High	Please provide XTRACT models (listed in Table 5-1 SGH report Volume 3) used to calculate Nonlinear Hinge properties.	ROH/LH	2019_01_30	Files transmitted	5/15/19: Resolved
118	1/30/2019	MIS	3		Please discuss how the Moment-Curvature diagrams (Figure 5-4 to 5-11 of SGH report Volume 3) would change if the square piles are modeled as square rather than round in XTRACT. An increase in initial strength may affect the moment and shear distribution in the pile, and the response mode (flexure vs. shear governed). Could this negatively impact the behavior?	ROH/LH	2019_3_06	If modeled as square, the moment-curvature behavior of the piles would exhibit an increased initial strength prior to crushing the unconfined concrete. We feel that modeling this initial strength is not appropriate for cyclic loading. LERA analyzed the piles with a square cross section. We compare their moment-curvature results to ours in the attached "Supplement No. 118". Based on the comparison, accounting for increased initial strength would not negatively affect pile behavior. Shear demands are likely to increase by about 50% but would remain less than the shear strength of the piles.	5/15/2019: Resolved pending review of results from foundation spring bounding analyses. 6/26/2019 - Resolved
119	1/30/2019	MIS	3		Please confirm which set of soil spring relations shown in Figures 4-4 to 4-7 of SGH report Volume 3 was used in the SAP2000 models of the individual piles (Section 5 of same report). Please tabulate the values used in the modeling of multilinear links representing soil, since it appears that some curves are overlapping in these figures.	ROH/LH	2019_3_06	We used the soil spring properties labeled "ENGEO_LB" and shown with solid lines in Figures 4-4 to 4-7. The attachment "Supplement No. 119" contains tabulated values for the springs' nonlinear force-deformation relationships we used in the SAP2000 pushover analyses of the piles. Each spring name contains the spring centerline depth below bottom of the mat in inches.	5/15/2019: Resolved pending review of results from foundation spring bounding analyses. 6/26/2019 - Resolved
120	1/30/2019	MIS	3		Please explain the basis for bilinearization of the Moment-Curvature diagrams shown in Figure 5-4 to 5-11. How do these bilinear models compare to the bilinearization done by XTRACT, and how may ignoring the peak strength change the composite foundation backbone and the predicted response?	ROH/LH	2019_3_06	The bilinearizations are similar to those done by XTRACT. In observing the Xtract bilinearizations, we observed that the "plateau" appeared a bit high to us, and we wanted to make sure we were not overly optimistic in estimating pile shear capacity. In the end, the plateau values we judgmentally selected were not very different from those recommended by Xtract. A sample comparison and further discussion is in the attachment "Supplement No. 120". Ignoring the peak strength obtained from moment-curvature analysis slightly reduces strength at low displacements under monotonic loading. We do not expect this to affect foundation response significantly. Please refer to our response to Comment 118 for discussion of the effect on increasing initial peak strength on the governing pile response (i.e. governed by shear or flexure).	5/15/2019: Resolved pending review of results from foundation spring bounding analyses. 6/26/2019 - Resolved
121	1/30/2019	MIS	3	High	Please provide a sample SAP2000 model (with nonzero initial pile head rotation) used for pushover analysis of individual piles.	ROH	2019_01_30	Model provided	5/15/19: Resolved
122	1/30/2019	MIS	3		Please explain the hinge length used in the definition of the plastic hinges of the piles in the SAP2000 model. Explain the basis for that assumption, as well as location and number of the plastic hinges along the pile. How sensitive are the results with respect to these assumptions?	ROH/LH	2019_3_06	We calculated plastic hinge lengths based on CBC 2016 recommendations. We located hinges at sufficient density that we are confident in the results from the model. Please refer to the attached "Supplement No. 122" for additional discussion.	5/15/2019 - Resolved. We recommend this discussion be included in the final calculations. 8/5/2019 : Resolved
							2019_06_07	Please refer to the updated Calculation Volume 3, Section 5.1 - revision 5.	
123	1/30/2019	MIS	3		What is the assumption for strength loss in the pile plastic hinges. Does a sudden drop to zero happen at the end of the models shown in Figure 5-4 to 5-11 (SGH report Volume 3)?	ROH/LH	2019_3_06	We assumed a sudden drop to zero hinge strength, and assumed the pile retains zero lateral load carrying capacity when any hinge exceeds its ultimate rotation capacity. This was only used in computing the lateral foundation backbones. Note that our analyses do not predict any pile hinges to actually reach strength loss.	5/15/2019: Resolved
124	1/30/2019	MIS	3	High	Provide a sample of pile group model in SAP2000 (Including overturning).	ROH	2019_01_30	Model provided	5/15/2019 - Resolved.
							2019_06_07	Please refer to the updated Calculation Volume 3 - revision 5.	
125	5/15/2019	GGD			Given that several of the analyses show higher force and deformation demands in the PG&E vault on the south side of the mat, please confirm whether the vault mat has been inspected for damage.	ROH/LH	2019_06_07	Based on discussions at the May 22, 2019, meeting with the EDRT, SGH will provide an observation report that includes photos of the PG&E vault slab located south of the tower.	8/5/2019 : Resolved
							2019_07_19	Please refer to the attached observation report Supplement 125.	
126	5/15/2019	SV			CSA March 1, 2019 memo presents data regarding differential settlement contours between 2009 and 2018. Data indicates substantial rigid body rotation toward Fremont street has occurred during this time period. Are data presented on Figure 14 consistent with pile head rotation assumptions used in dynamic response analysis? Please confirm as discussed in meeting on 4.23.3019.	ROH/LH	2019_06_07	The pile head rotations and mat deformed shape assumed in our analyses are based on differential settlement occurring between the surveys performed May 4, 2009 and June 2, 2017. We have compared differential settlement contours between 2009-2017 with those between 2009-2019 and observe similar rigid body rotation. Please refer to Supplement No. 126 for a side-by-side comparison of differential settlement contours.	6/26/2019 - Comment Resolved
127	5/15/2019	SV			The monitoring Program outlined below was presented in an e-mail dated March 22, 2019. Please confirm if this is the current intent and if the program has been incorporated in the latest design drawings and specifications. • 21 instances of monitoring of the basement and exterior piezometers and extensometers for two years. This includes travel time and costs, time on site and a plot and is broken down as follows: o 6 months of bi-weekly readings (13); o 1.5 months between monitoring (3); and o Quarterly monitoring for 15 months (5). • 74 instances of prism and basement monitoring review and analysis based on the following schedule: o 9 months of weekly monitoring immediately prior to, during, and following construction (39); o 3 months of bi-weekly monitoring (6); o 1.5 months between monitoring for two years (16); o Quarterly monitoring for two years (8); and • Annual monitoring for five years (5). Please confirm as discussed in meeting on 4.23.3019.		2019_06_07	These criteria are listed on the new sheet on Structural Monitoring.	6/26/2019 - Comment Resolved

Comments by EDRT						Responses by Design Team			Date
128	5/15/2019	SV			Referring to proposed monitoring program (Comment 127), please (a) clarify how and at what points along the mat the monitoring of the mat extension will be performed, (b) establish a safe limit of vertical movement for the mat extension, and (c) plan of action if the measured uplift displacement of the mat approaches the safe limit. In addition, rather than presenting the period of weekly monitoring as an absolute number of readings (39), indicate how many weeks before and after construction monitoring is intended to occur on a weekly basis, require weekly monitoring during construction, and define construction milestones that are considered to be the start and end of construction for the purpose of the monitoring program. Since there may be revisions to the construction schedule as the project develops further, an approach tied to milestones would seem to have less ambiguity. Please confirm as discussed in meeting on 4.23.3019.			These criteria are listed on the new sheet on Structural Monitoring.	6/26/2019 - Comment Resolved
129	5/15/2019	SV			Revision 2 - Volume 2, Page 43, top of page: correct missing section reference.	ROH/LH	2019_06_07	We corrected this reference in our revised analysis package, Volume 2, Sections 1.4 and 2.6.1, Revision 5	6/26/2019 - Comment Resolved (note revised Vol.2 shows a missing section reference in 1.4 on pg. 40)
130	5/15/2019	SV			Please confirm that during an MCEr event, the two rods connecting the mat extension to the top of the pile (i.e. the fuse) can sustain any yielding that occurs as a result of nonuniform propagation of vertical bedrock motion up to the mat.	ROH/LH	2019_06_07	From our email correspondence with John Egan we understand that the expected MCE_R peak differential displacement between the rock and the bottom of the Colma formation will not exceed 2 inches based on site response analysis performed by Slate Engineers. The rods connecting the mat extension to the top of the pile are detailed for a total displacement of 6.4 inches = 0.05 x (10'-8"). Testing of a rod demonstrates that substantially more strain capacity exists.	8/5/2019 : Resolved
	6/26/2019				Please provide MCEr peak vertical displacement at the bedrock level. This displacement will directly be transmitted to the rods.		2019_07_19	We compute peak vertical displacement at the top of rock by converting the peak horizontal displacement obtained for the site response analysis to horizontal using the Abrahamson & Gulerce (2011) V/H relationship. This results in peak estimated displacement at the top of rock of 12.9 inches. Assuming that the mat does not move, this results in a strain of approximately .072 (12.9"/(14.75*12) over the free length of the rods. We note that current 95th percentile estimates of settlement over the next 40 years is approximately 1-1/2 inches at the extreme east of the mat along Mission and extreme south end of the mat along Fremont. This results in an additional 0.2% strain for a total strain of approximately 0.075. We note that the rod material has been successfully tested statically to a strain of 0.23, which is substantially in excess of the projected total strain. Regardless, we acknowledge, given the cyclic (though not fully reversed cyclic) nature of the seismic loading, that there is a small possibility of fracture of some rods, given MCE shaking. This would not result in building instability, but would require repair.	
131	5/15/2019	CSS	14		Please clarify/modify Specification 31 6329 to address the following: 1) strength for soil/bentonite mixture in specification (500-750 psf) does not match Sheet T002 (500 psf max.); 2) complete sentence on 3.04C1; 3) Section 3.04E - we believe waiting 72 hours to place concrete after approval of rock socket is too long - suggest reducing to 12 hours; 4) Section 3.09C2 - clarify how the condition of the rock socket sides and bottom will be observed since drilled shaft will be filled with drilling fluid.	ROH/SKH	2019_06_07	We have revised the drawing to match the specification with regard to the cement/bentonite mixture strength. (2) Please see response to Comment 18 (3) Please see response to Comment 18 (4) Drilling fluid will not be used in the RCD drilling method. The drilling will be accomplished with water, which is extracted through the central drill shaft along with the cuttings. We are not relying on any end bearing of the shafts and do not expect to mobilize any end bearing due to the softened material remaining at the bottom of the shafts. The sides of the rock sockets will be observed via down-hole video.	8/5/2019 : Resolved
132	6/26/2019	MS, SV			In Volume 3, a new torsional spring was introduced for the existing foundation behavior (Section 6.1.3, page 159). Compared to the yield rotation of 0.0006rad, the rotation demand of 0.025rad reported on slide 44 of the May 22 presentation seems large (0.025*100'*12" = 30"). Perhaps this is a typo and the intent is to report 0.025% (=0.00025 rad). Please confirm the expected rotations and report the resulting displacement at the corners of the mat. If the displacements are significantly larger than the reported displacements at the center, please comment on the effect of the displacements on the response of existing and retrofit piles. For the existing piles, please confirm how the retrofit will affect the existing piles in the south-east corner (i.e., will the retrofit increase or decrease lateral deformations of the		2019_07_19	Agreed this is a typo. Maximum rotation in the retrofit condition is about 0.025%, or 0.00025 rad. See Supplement No. 132 for maximum mat corner displacements in the existing and retrofit conditions. The retrofit reduces lateral displacement of existing piles at all locations.	8/5/2019 : Resolved
133	6/26/2019	MS			For the mat seismic loading verifications (Volume 3, Section 7.7.1.2.2, page 261 and following), please confirm what the arrows with annotated DCRs on the plots mean. For example, Figure 7-24 on page 269 shows various orange and red		2019_07_19	We use the arrows to indicate that the DCRs are developed by adding the demands and capacities of adjacent design strips to develop the reported DCR. We do this because we do not expect the mat to fail over a single strip width, and high moments will redistribute to the	8/5/2019 : Resolved
134	6/26/2019	MS			Sheet S001 Foundations, Note 1 - please update the reference to the current (final) geotechnical report		2019_07_19	Please refer to the updated sheet S001.	8/5/2019: Resolved
135	6/26/2019	MS			Sheet S001 : Please specify material (product) specifications for the Sonotube		2019_07_19	Detail 1/S502 specifies Sonotube® for this application. Supplement No 135 provides the manufacturer's specification for this material. It contains no particular technical detail on the material.	8/5/2019: Resolved, but please revise section detail 5/S501 to show the Sonotube.
						SKH	2019_08_05	We revised 5/S501 to show the Sonotube	
136	6/26/2019	MS, GD			We do not see any provisions made for filling the annulus between the outer and inner casing of the upper region of the pile after jacking (e.g., Detail 9 on S101)? Please specify access and procedure for grouting, including provisions to check for (avoid) the formation of voids in the grout.		2019_07_19	The annulus between the outer and the inner casing is to be filled from the bottom up using a tremie method. We specified the CLSM to have a higher specific gravity than water and to be highly flowable. We therefore do not anticipate any voids of significant size to occur given the nonstructural purpose of the fill.	8/5/2019: Resolved
137	6/26/2019	MS, GD			(1) Please explain the purpose of the interior cross walls in the jacking vault (S203 and 2/S302), (2) Clarify the intent of the note "dowels to match wall reinf., typ" (is this meant to refer to the vertical bars or also to the horizontal bars with anchors into the existing basement walls) and confirm the detail for how the cross walls (both the interior and end walls) are connected to the existing basement wall.		2019_07_19	(1) The interior cross walls behave as shear walls to transfer lateral load to the mat. Their design is governed by the MCE soil pressures. The vault top slab behaves as a diaphragm to transfer this lateral load to the crosswalls. (2) That note is intended to refer to the vertical bars in the wall. The leader from this comment has been adjusted in the revised drawings for clarity.	8/5/2019: Resolved
138	6/26/2019	MS, GD			On S302, please confirm if the epoxy anchor embedment of the #6 bars is intended to fully develop the bars, and If so, whether 6" is sufficient embedment to do so.		2019_07_19	The #6 bars are not intended to be fully developed by the 6" embedment depth. Please refer to the epoxy anchor check starting on Calculation Package Volume 4, Revision 4, Page 89 for the capacity check of this connection.	8/5/2019: Resolved
139	6/26/2019	MS, GD			Are piezometer data from the new basement boring instrumentation available, and are these consistent with other measurements that are being used to establish the updated settlement calculations? We would suggest comparing any new data with the assumptions in the current settlement model.		2019_07_19	Refer to Supplement No. 139 for a plot of readings from the basement piezometers. The ground water elevation used in the settlement calculations reflects this updated understanding of ground water behavior.	8/5/2019: Resolved
140	6/26/2019	All			As we understand the proposed plan, upon completion of the project the intent is to have two sets of boring instrumentation to monitor strains and water pressure. One set is located in the basement near Mission street, and the second is located under the new mat extension on Fremont street (near column line C, T001). Please confirm when the instrumentation will be installed in the Fremont street boring and what precautions are being taken to prevent damage during construction. To the extent that reliable instrumentation will be important to help resolve future questions (e.g., if future settlements differ from the estimates), it is suggested to install redundant sets of instrumentation in one or two of the other planned borings on Mission and/or Fremont street (T001).		2019_07_19	The drawings have been revised to show a total of 4 exterior geotechnical monitoring probes, 2 along Mission Street and 2 along Fremont Street. These will be placed in the geotechnical pilot boring holes. One each along Mission and Fremont will have piezometer and one each extensometers. The instruments will target the upper 50 feet of OBC deposits.	8/5/2019: Resolved pending confirmation (see new comments #168, 178 and 17) of (a) details of instrumentation, (b) information on when it will be installed, and (c) provisions to protect the instrumentation during construction. 8/26/2019: Resolved
141	6/26/2019	MS			The monitoring plan on S207 indicates that monitoring will continue for 5 years, but DBI Information Sheet IS S-18 requires 10 years for new construction. Please check and confirm.		2019_07_19	The newly established 10-year criterion applies to new buildings. If required by DBI we will extend the monitoring period out to this duration.	8/5/2019: Resolved, per drawings revised to show 10 year monitoring
142	6/26/2019	MS			See marked up copy of S401 and S402 for some suggested improvements to reinforcing bar layout.		2019_07_19	Please refer to the updated sheets S401 and S402.	8/5/2019: Resolved

Comments by EDRT						Responses by Design Team			Date
143	8/5/2019	SV	23		Section 1.0, Page 1/24, Second Bullet - Does the objective of arresting primary consolidation apply to the soil under the entire building footprint?	JE	2019_08_07	Our analyses indicate that a vast majority of the Old Bay Clay (OBC) will be returned to an overconsolidated stress state with implementation of the Perimeter Pile Upgrade and primary consolidation of these soils will be arrested. A small portion of the OBC near the southeast corner of the Tower footprint will remain normally consolidated, but the state of stress will be significantly reduced, to the extent that future settlement (between the present and 2060) due to primary consolidation in this small area is expected to be less than an inch.	8/26/2019: Resolved
144	8/5/2019	SV	23		Section 7.0, Page 7/24 - The sentence " if sand lenses remain susceptible to liquefaction..." As stated in the later part of this section, It is expected that vibration caused by pile driving at close spacing has mitigated the potential for soil liquefaction. Please clarify.	JE	2019_08_07	References to sand lenses remaining potentially susceptible to liquefaction have been removed from the revised geotechnical report. The liquefaction susceptibility of sand lenses in the Upper YBM and in the Marine Sands is judged to be negligible as a result of densification from pile driving.	8/26/2019: Resolved
145	8/5/2019	SV	23		Section 8 & Appendix B - Has the effect of building tilting on distribution of gravity loads been considered in settlement analysis?	JE	2019_08_07	The existing gravity load distribution utilized in the settlement analyses was provided by SGH; although, analyses were performed to evaluate effects of the moment induced by tilting rotation on the load distribution and that effect was found to be rather small. This response is included in Section 8.0 of the geotechnical report.	8/26/2019: Resolved
146	8/5/2019	SV	23		Section 8, Page 8 - ASCE/SEI 7-16 or ASCE/SEI 7-10? Please clarify	JE/ROH	2019_08_07	The project Basis of Design in Volume 1 of the calculations is revised to show that design is in accordance with ASCE 7-10, with some exceptions, where ASCE 7-16 is used to take advantage of updated science and industry development. The ground motions were developed primarily following the procedures of ASCE/SEI 7-16 which represents the current generation of seismic source characterization and ground motion prediction equations. The final recommended spectra was checked against ASCE/SEI 7-10 for final conformance with the 2016 San Francisco and California Building Codes that are currently enforced by the City and County of San Francisco Department of Building Inspection. Please see the revised geotechnical report for this change. Please also see Supplement No. 146	8/26/2019: Resolved
147	8/5/2019	SV	23		Section 8.1, Page 9/24 - What would be the increase in the "fuse" rod axial loads if building settles 8.0 inches at the PPU pile locations?	ROH?JE	2019_08_06	Based on testing of sample rods of similar specification, we anticipate a pile force at a settlement of 8 inches of 1,200 kips. Note that we have evaluated the adequacy of the mat for this force. To the extent that the new PPU piles will experience increased load as secondary compression continues, this will tend to further reduce the OCR and reduce the amount of secondary compression that occurs. This ancillary effect on settlement has not been considered in the settlement analysis, and results in conservative estimates of post-retrofit settlement. It is important to note that at the locations of the new piles, future settlement is projected as negligible. It is extremely unlikely that settlement on the order of an additional 8 inches would ever occur at the locations of these new piles.	8/26/2019: Resolved
148	8/5/2019	SV, GD	23		Section 8.1, page 9/24 - The sentence "...,with more tilt correction resulting the more this settlement occurs over time from 2020 to 2060" is not clear. Have you calculated estimates of the expected amount (range) of tilt correction? Please clarify.	JE	2019_08_07	Language to clarify this comment has been added to the geotechnical report. Tilting of the tower to date has closely approximated by rigid body rotation and we anticipate this behavior will continue. Review of our projected settlement profile with time (Figure 8) and consideration of the variability associated with this prediction determined in the Monte Carlo Analysis (Appendix B) suggests that tilt correction will not fully bring the tower back to its original vertical position	8/26/2019: Resolved
149	8/5/2019	SV	23		Section 8.2 & Appendix C - Appendix C states that "We understand that SGH will model the axial load-resistance characteristics of the existing piles as a single spring in the vertical direction of their structural model". Is this correct? We understand multiple vertical soil springs are attached at the intersection of grillage representing the mat.	JE/ROH	2019_08_07	The structural model replicates the estimated distribution of capacities for the existing piles and nonlinear pile stiffness both with regard to vertical and lateral stiffness, so as to capture rotational effects. The structural model does not include a 1 to 1 correspondence between modeled new piles and existing piles, but rather represents the existing piles with elements at the 5-foot nominal grid spacing used to model the mat foundation. Scaled spring stiffnesses of these existing piles are modeled on a tributary area basis, based on the capacity curves estimated herein for piles in different regions of the mat.	8/26/2019: Resolved
150	8/5/2019	SV	23		Section 8.4.1, Page 12/24 - It is stated that "..., frictional resistance will also be developed between the base of the podium mat and the underlying bearing soils,...". We understand that the Podium is held down with tie-downs to resist hydrostatic uplift pressure. As such, there is no contact at the base of the podium and underlying bearing soils. Please clarify.	JE	2019_08_07	Original project plans indicate that tie-down anchors were used on the west side of the Podium structure under the vehicular ramp to resist the potential for hydrostatic uplift pressure. The effective frictional resistance in this area may be small or negligible. Other areas of the Podium mat impose positive weight and contact with the soil and will develop frictional resistance with the underlying soil.	8/26/2019: Resolved
151	8/5/2019	SV	23		Section 8.4.1, Page 12/24 - If only frictional resistance against either the north or south Podium basement wall is considered, the effective normal stress would be higher than the at-rest pressure. Please explain.	JE	2019_08_07	During earthquake ground shaking, the earth pressure on the basement walls will vary temporally due to the transient and cyclic nature of the ground movement. At any given point in time during the shaking, the earth pressure controlling the frictional resistance may be between zero (no contact) and potentially a passive condition. If relative ground movement has caused it to be zero on one basement wall face, it likely is greater than at-rest on the face of the opposite wall; however, the magnitude of that earth pressure is indeterminate, so estimating it as at-rest is considered as conservative. Please see the revised geotechnical report Section 8.4.2 for this change.	8/26/2019: Resolved
152	8/5/2019	SV	23		Section 9.3, Page 17/24 - Installation of jet grout columns results in ground heave and spoils that may be equal in volume to the volume of jet grout Column. Please address the effects of heave on adjacent streets.	JE	2019_08_07	Further discussion of the potential for the jet grout columns to result in lateral soil movements and ground heave in areas adjacent to the installation has been incorporated into Sections 9.3.1 and 9.3.2 of the revised geotechnical report.	8/26/2019: Resolved
153	8/5/2019	SV	23, 22, 20		Section 9.4, Page 19/24 - In case the water seeps into the excavation between jet ground columns or at the contact between jet grout columns and the base plug, the contractor should be ready to seal the leakage area using an appropriate method (e.g. use of pressure grouting equipment). This should be addressed in the PPU Specification (document 22) and/or drawing notes (document 20).	SKH/JE	2019_08_07	Chemical grouting to address water leakage in the shoring system is specified in Section 31 50 00, Article 3.09 "Control of Water Leakage" Language has also been added to the geotechnical report to this effect.	8/26/2019: Resolved
154	8/5/2019	GD	23		Appendix B: We appreciate the use of Bayesian Inference to refine the settlement calculations based on measurements of mat settlement in 2011. Please report how well the updated model predictions compare with the current year (2019) measured settlements, since this is presumably accounted for in your estimate of the expected additional settlement between 2020 and 2060 (after installation of the foundation upgrade).	JE	2019_08_16	See Figures B-13 and B-20 (Rpt Rev 1) for the calculated settlement results compared to measured 2019 data before and after Bayesian updating, respectively.	8/26/2019: Resolved
155	8/5/2019	GD	20, construction		Given the close proximity of pile driving and other construction activities adjacent to the tower, have you considered requiring temporary protection for the exterior curtain wall of the tower? Suggest that this be mentioned in the New Construction note on S001.	SKH	2019_08_05	We added a note on S001 indicating that protection of the existing façade is required.	8/26/2019: Resolved
156	8/5/2019	GD	20		Please show Sono Tube in Section 5/S501. Also, on S502, the section call out is incorrectly indicated as 3/S501; so please correct to 5/S501.	SKH	2019_08_05	We added Sonotube to the detail and changed the Section references.	8/26/2019: Resolved
157	8/5/2019	GD	20		S205, Please correct spelling typo, note 4 "pile lengths"	SKH	2019_08_05	Spelling corrected.	8/26/2019: Resolved
158	8/5/2019	GD	20		S204, We would suggest including a few more survey points to monitor during pile jacking on the mat adjacent to the perimeter walls on the Mission St and Fremont St sides (similar to SM 7).	SKH	2019_08_05	We included eight additional monitoring points, all existing, on S204 adjacent to the perimeter walls and have also shown these points on the monitoring plan, S2.07	8/26/2019: Resolved
159	8/5/2019	GD	20		S401, Suggest adding a note (and possibly revising drawing line work) to the sections on S402 to refer to the water proofing detail around the new piles as shown on S503.	SKH	2019_08_05	We updated the graphics on S402 and added notes referring to S503 for waterproofing	8/26/2019: Resolved
160	8/5/2019	GD	21		T002, Note 2 under Pile Testing indicates that the plan is to demonstrate the ultimate skin friction of 10 ksf in the Franciscan Formation. However, the Geotechnical Report (pg. 14 of 24) indicates that the skin friction strength in the first 10 ft or so of weathered rock is expected to be 3 ksf. Please clarify this note and confirm that the o-cell locations are consistent with the assumed rock strengths to obtain the desired outcome of the tests.	SKH	2019_08_05	The 3ksf strength of the weaker upper Franciscan Formation is considered in all of our calculations. We now plan 3 O-cells. The lowest of these will be 10ft above the bottom of the 38-foot long socket into rock. We anticipate failing this down, with a target load of 540k. The resisting load will consist of the 28 feet of rock socket above plus, whatever length of Alameda and Old Bay Clay exists above. We estimate a resistance on top of the O-cell of (540 in the shaft between O-cells 1 and 2; plus 436 between O-cells 2 and the top of solid rock; 141 k in the weathered rock, and 500k in the Alameda formation and Old Bay Clay = total of 1500k resistance above O-cell 1). O-cell 2 will (we hope) fail the shaft between O cells 1 and 2 downward with 540 kips of push down reacted by 950k of resistance above. O-cell 3 will be tested with Ocells 1 and 2 locked off. It will have resistance of 1626 downward, and we hope will have resistance of 521k upward against the weak rock, Alameda and Old Bay Clays.	8/26/2019: Resolved
161	8/5/2019	MS,GD	3, 23		The final geotechnical report states (Page 12, Section 8.4.1, third paragraph): "...it is recommended that the lateral resistance for a single basement wall be considered at any given time..." Calculations Volume 3 (rev 5), page 157 indicates that this resistance is considered in the structural analysis model, but please clarify whether the eccentricity of the resistance is considered.	ROH	2019_08_06	Upon review, we believe the recommendation that soil friction be considered to act against only one wall of the building at a time was excessively conservative, as well as impractically difficult to model. The intent of this recommendation was to recognize that as shear waves pass through the soil, pressure would increase against one wall (wall 1) and simultaneously decrease against the opposite wall (wall 2). Due to the significant depth of the podium basement, the at-rest pressures against both walls 1 and 2 are substantially larger than the dynamic increment. In our structural model the lateral resistance due to friction is taken as that resulting from the at-rest pressures, assumed to act simultaneously on both sides of the structure. While minor eccentricities in lateral pressure will occur as shear waves pass through the site, these would have small effect on overall resistance. It is worth noting that under our base case stiffness assumptions for soil springs, maximum base displacement of the tower towards the podium for any of the ground motions is 1-1/4 inches. Since the friction is assumed to act against the podium walls only (another conservative assumption) and frictional resistance against the podium walls are not mobilized until 1) 1 inch gap between the structures close, and 2) 0.1 inch of podium displacement occurs, side wall friction is only active for the final 1/8 inch of displacement in the worst case ground motion considered. Considering the small eccentricities in this frictional resistance, and that these are constantly changing, would have negligible impact on predicted building response.	8/26/2019: Resolved

Comments by EDRT						Responses by Design Team			Date
162	8/5/2019	MS	23		During our review of the geotechnical report we came across the following editorial and typographical issues that we recommend be addressed in the final version of the report: • Page 6, second paragraph, second sentence: Typo, remove duplicate words "[...], groundwater [...] was at a depth of approximately ~~a depth of~~ 3 to 4 feet [...]." • Page 11, Section 8.3, first paragraph, last sentence: Typo, "SGH is performing a non-linear structural evaluation which **including** the time histories at the bottom of the foundation mat (~25 feet bgs)." Suggest changes to **applies**. • Page 11, Section 8.4: This section starts rather abruptly. Should there be a leading sentence to introduce the resistance from bearing of the basement walls and foundations on the surrounding soils? "Combined lateral load-displacement curves" could also include the existing and new piles. • Page 13, second paragraph, fourth sentence: "Once the OBC **later** is encountered, [...]" Suggest changing to **layer.** • Page 13, fourth paragraph, fourth sentence: "Upon **completion** the CIDH Pile jacking, [...]" Suggest changing to either "Upon **completion of** the CIDH Pile jacking, or "Upon **completing** the CIDH Pile jacking". • Page 13, Section 9.1.1, first paragraph, first sentence: "The axial pile capacity presented in this section are based on [...]" Suggest changing to "...pile capacity [...] **is**" or "...**pile capacities** [...] are...". • Page 15, Section 9.1.2, third paragraph, first sentence: "The large vertical demands on the rock socket and the desire to measure [...]". Suggest adding **Given** at the start of the sentence. • Page 15, Section 9.1.2, third paragraph, second sentence: "The bi-directional static load tests s5ould be positioned at two or more locations within the length of the rock socket to **all** for interpretation of the skin friction [...]". Suggest changing to **allow.** • Page 16, Section 9.1.3, first paragraph, second sentence: "[...] capable of drilling to the required depths and **though** a variety of material strengths.". Suggest changing to **through.**	JE	2019_08_07	These editorial suggestions have been incorporated into the revised geotechnical report.	8/26/2019: Resolved
163	8/5/2019	MS	23, 3		Please confirm whether the existing basement has been checked for construction loads (geotechnical report page 16, section 9.2)?	LVH	2019_08_06	We confirmed the adequacy of the basement walls for lateral soil pressure associated with the pile installation equipment in combination with existing static earth pressure. Please refer to calculations in Revision 4 - Volume 4, Section 5 of the May, 2019 final analyses submitted to the EDRT.	8/26/2019: Resolved
164	8/5/2019	MS	21	high	Sheet T001: Clarify if instrumentation at three of the exploratory borings is installed under this permit, or during construction of the upgrade. No details of the instrumentation are provided on these drawings, but it is assumed the instrumentation would be installed shortly after completion of the borings, so the holes are not kept open for an extended period.	SKH	2019_08_05	Instrumentation of the geotechnical borings (4 total) is indicated on S207, which is part of the Perimeter Pile Upgrade permit package. The two permit packages will be implemented concurrently and will be under the same construction contract. Although these are two packages for permitting, there are cross-references between the sets. Details of the instrumentation are to be provided by the instrumentation supplier (GeoKon). S2.07 has been revised to required installation of instrumentation within 24 hours of boring operation completion. Also see response to Comment 174.	8/26/2019: Resolved
165	8/5/2019	MS	21	high	Sheet T002, Drilled piles, item 1c: It is inconsistent with the installation of the production piles to fill the annulus between the outer casing and shaft liner before placing the pile concrete and pile jacking. It would also change the behavior of the pile in the pushup testing from the o-cells at the pile tip. Also note that all details show this annular space as "void", and no tremie pipes are shown. Grouting of the void as part of the testing program/mockup would be recommended but doing it after the "mat extension mockup block testing" would appear to be more consistent.	SKH	2019_08_05	We have deleted Item 1c from the pile notes on T002. It is our intent to leave the annular space open during testing. Because the pile will be tested using O-cells near the base of the pile, there is no need to fill the annular space - this portion of the pile will not be loaded. Note that this is consistent with the production pile condition, where the piles will be loaded prior to filling the annular space.	8/26/2019: Resolved
166	8/5/2019	MS	21		Sheet T002, Drilled piles, item 7b: Clarify what "point ***on*** interest" means. Would this be the pile tip elevation?	SKH	2019_08_05	We have revised this note to read "any point at depth." The intent of the more general wording is that the limit should apply at any point along the pile as well as at the tip.	8/26/2019: Resolved
167	8/5/2019	MS	21		Sheet T002, Pile concrete & reinforcing steel: Clarify what the requirements for the CLSM are. Note that the geotechnical report requires a specific gravity of 1.1 of greater (page 13, 4th paragraph).	SKH	2019_08_05	Since the CLSM will not be employed in the indicator pile, there is no need to specify its properties on this sheet.	8/26/2019: Resolved
168	8/5/2019	MS	21		Sheet T002, geotechnical investigation: Clarify that borings will be backfilled. Also, correct typo in item 6, "Advance additional soil boring at ~~to~~ the location shown..."	SKH	2019_08_05	We have corrected the typo. We have added a Note 7 indicating that the instrumentation is to be installed according to Sheet S207 and that borings shall be backfilled.	8/26/2019: Resolved
169	8/5/2019	MS	21		Clarify when the mat extension mockup block should be constructed. Assume the pile is not connected to the jacking beam when the pile testing engaging the o-cells is carried out?	SKH	2019_08_05	Mat Extension Mockup Block Note 1 on Drawing T002 indicates that the mat extension mockup block testing is to occur following the pile testing. We have added to this note to indicate that construction of the mockup block is to occur after the pile testing.	8/26/2019: Resolved
170	8/5/2019	MS	21		Recommend adding a note clarifying that it is the contractor's responsibility to provide temporary shoring to allow construction of the mat extension mockup block. Should this shoring be removed to assure it won't impact the main construction? Also, clarify that demolition and removal of the mockup block is required.	SKH	2019_08_05	We have added the recommended notes to Drawing T002	8/26/2019: Resolved
171	8/5/2019	MS	20		Sheet S001, Concrete & reinforcing steel, item 6: Is the 150 psi lean concrete mix the same as the CLSM called out in detail 4/S501? Note that the geotechnical report requires a specific gravity of 1.1 of greater (page 13, 4th paragraph).	SKH	2019_08_05	This material is not the same as the CLSM and is not required. We have removed this material from the list of concrete mixes.	8/26/2019: Resolved
172	8/5/2019	MS	20		Sheet S101, step 5: The title still reads "Jet **grut** plug" rather than "Jet **grout** plug".	SKH	2019_08_05	We have fixed the typographic error.	8/26/2019: Resolved
173	8/5/2019	MS	20		Sheet S101 & S102: Please clarify where in the construction sequence the annular space between the 36" outer casing and 24" inner casing will be filled with the CLSM shown in detail 4/S501.	SKH	2019_08_05	We have revised the graphics on S411 and included filling of the annular space in Step 10 on S102.	8/26/2019: Resolved
174	8/5/2019	MS	20	high	Sheet S200: Show instrumented borings to be protected in place during demolition.	SKH	2019_08_05	We have added notes requiring protection of the instrumented borings to S200.	8/26/2019: Resolved
175	8/5/2019	MS	20	high	Sheet S202 & S203: Show locations of instrumented borings to be protected in place during construction. Also, provide a detail on how the instrumentation is terminated in the mat extension. Will this detail allow for differential movement between the mat and the instrumented boring during jacking?	SKH	2019_08_05	We added the locations of instrument borings and the indication of required protection on Sheets S2.02 and S2.03. Termination of the instrumentation within the mat extension is provided by the supplier. The extensometer instruments are able to accommodate the jacking movement. Notes have been added to drawing S2.07 requiring that piezometer leads accommodate 2 inches of movement. Only 1/2 inch is anticipated.	8/26/2019: Resolved
176	8/5/2019	MS	20		Sheet S401: Please confirm the projection of the existing canopy along Fremont Street was considered. Cladding system fabrication drawings show the canopy projecting 7'-1" from grid line 1, which would align with the centerline of the pile. Is the intent to temporarily remove and reinstall the canopy where new piles are to be installed?	SKH	2019_08_05	Confirmed. The Contractor has an allowance in the budget for this work. We have added a note to Step 2 on Sheet S101 to clarify.	8/26/2019: Resolved
177	8/5/2019	MS	20		Sheet S501, detail 4, pile section - at temporary CLSM: Please clarify what is "temporary" about the CLSM. Is this flowable grout placed before or after construction of the vault (see earlier comment on construction sequence)?	SKH	2019_08_05	We have removed the word "Temporary." This title was originally referring to the temporary casing, which has now become permanent. The CLSM is installed following jacking of the piles, which we expect will occur prior to construction the top of the vault. However, this is the contractor's choice.	8/26/2019: Resolved
178	8/5/2019	MS	20		Sheet S501, detail 5, pile section within mat extension: Is it necessary to use steel pipe for the tremie pipes that fill the annular space between the inner and outer casing, or would PVC pipes work for these sacrificial tremie pipes? Are they stabilized along their length with spacers? How are pipe sections joined? Also, consider that jacking may cause the mat extension to slightly uplift; assume the tremie pipes can move freely?	SKH	2019_08_05	Given the significant length of the tremie pipes, we feel more comfortable with steel rather than PVC. These pipes are nor stabilized within the annular space - movement of the pipes can occur around the annulus, but is not harmful and is allowed. Joining of the pipe sections is the contractor's option as part of the means and methods of the Work. The pipes will travel upward during jacking along with the mat extension. This is acceptable, since the piles are not restrained at the bottom.	8/26/2019: Resolved
179	8/5/2019	MS	20	high	Sheet S503: There appears to be a conflict between the location of the tremie pipes and the clamp plates for the waterproofing. Clarify how the waterproofing is sealed around the tremie pipe and relief pipe, and how far the relief pipe needs to extend down below the mat to enter the void. Is protection of the void during construction of the mat extension considered means and methods? Or is the intent to clamp the waterproofing to the outer casing?	SKH	2019_08_05	We have corrected the inconsistency. We had incorrectly shown the waterproofing connecting to the 24-inch shaft liner. We have corrected the drawings to show the waterproofing connecting to the 36-inch outer casing. The temie pipe and the relief pipe are inside of the waterproof envelope. The relief pipe extends only to the bottom of the mat extension.	8/26/2019: Resolved
180	8/5/2019	MS	20		Sheet S503: Please clarify how the waterproofing terminates along the vault walls. Does the detail used at the roof turn and run down the walls, or is there a different blind installation detail? Also, is there a special detail at the transition from this vertical face to the CETCO TW-4 termination at the bottom of the mat?	SKH	2019_08_07	The waterproofing system for the vault has been revised to include the use of a crystalline waterproofing admixture in the concrete, e.g. Caltite, together with water stops at construction joints and a flashing/bentonite plug system at the entry of the piles into the base of the pile cap extension. Please refer to Sheet 5.03 for these details.	8/26/2019: Resolved, resolved, with the understanding that the design documents will be updated to incorporate vault waterproofing requirements prior to construction
181	8/5/2019	MS	23, App B		Appendix B, Page 1, second paragraph, last sentence: This sentence indicates that the 800 kip jacking force is applied to the pile ***prior*** to connecting the new pile to the existing mat. This is incorrect, as the weight of the building through the mat extension will be used to impose the jacking force. Please correct and confirm this does not affect the calculations.	JE	2019_08_07	This discrepancy has been revised to state that the jacking force will be applied to each CIDH pile head using the mat extension and the Tower as a reaction mass. This does not affect the calculations.	8/26/2019: Resolved

Comments by EDRT						Responses by Design Team			Date
182	8/5/2019	MS	23, App B		Appendix B, Page 2, last paragraph, first sentence: This sentence indicates that the pile loading is assumed to remain near the jacking load of 800 kips. Approximately 1 inch of rebound is predicted. How much would that change the jacking load, and is it reasonable to ignore? Is there a criterion for the loss of load where re-jacking is required?	JE	2019_08_16	As noted, the geotechnical settlement calculations assume a constant 800 kip/pile jacking force. The interaction between jacking, rebound, and ultimate settlement, is quite complex. The pile/loading rod system stiffness results in approximately 1 inch of system compression under 800 kips of jacking. Clearly if the system were to experience a full rebound of 1 inch, corresponding to estimated soil rebound associated with reducing the stress on the Old Bay Clay, this would completely unload the initial jacking force, in which case the building would return to the current settlement profile. However, this assumption does not include the additional secondary compression that is on-going simultaneously. As the building settles during this process, any lost jacking load in the piles would be recovered until an equilibrium condition is established. Using 1D consolidation theory incorporating varying rates of rebound due to the load transfer from the soil to the retrofit piles and varying rates of secondary compression based on overconsolidation ratio, the jacking load in the piles may increase somewhat until an equilibrium is established between the rebound and secondary compression. Regardless, the jacking plan (Sheet S2.04) now shows a note requiring rejacking upon loss of 100k. We intend to monitor this and will adjust this plan, as appropriate, when the actual settlement behavior post-jacking is observed and processed, although we do not anticipate that rejacking will be necessary.	8/26/2019: Resolved
183	8/5/2019	MS	23, App B		Appendix B, Page 3, Section 4; page 14, Section 5.3.7 and section 5.4.2 discuss the process of validating the model and updating the distribution resulting from the 6500 results based on comparison of model predictions for 2011 to measured settlements. Please clarify if this process includes only one point on the basement mat foundation (marker SM-27) or all 24 settlement markers. Similarly, clarify if Figures B-15 is based on 6500 scenarios at marker SM-27 or all 24 points over the building footprint. An earlier report (dated November 30, 2018) provided results for two points (notably with a distinct difference in predictive quality), and the report (page 4, first paragraph) indicates 24 points were tracked through the calculation. If only one point is used, it is recommended that the impact of using all points is investigated. If all points were used, please provide results for additional points and propose a measure of quality of fit for the predicted settlement over the entire mat surface (rather than a single point).	JE	2019_08_16	The validation and Bayesian updating is based on all 24 settlement markers. Figure B-18 (Rpt Rev 1) is based on the scenario with mean parameters compared to the 24 settlement markers. The results presented measure quality of fit over the entire mat surface based on fit to all 24 settlement markers.	8/26/2019: Resolved
184	8/5/2019	MS	23, App B		Appendix B, Page 3, Section 4, last paragraph: the EDRT cannot task the design team. It is recommended that reference to the EDRT be removed.	JE	2019_08_07	Reference to the EDRT has been removed.	8/26/2019: Resolved
185	8/5/2019	MS	23, App B		Appendix B, Page 9, Section 5.3, last paragraph before subsection 5.3.1: Please clarify how many one-dimensional columns the 8ft spacing results in (~140?). Also, how are results interpolated for comparison with observed settlements assuming the grid of soil columns and the settlement markers do not align perfectly.	JE	2019_08_07	Our FLAC model extends beyond the footprint of the mat and includes a total of 313 columns, with 247 of these columns located within the mat footprint. The columns outside the mat footprint are used so that results at the boundary can be interpolated to fit the building footprint. To validate the statistically estimated soil parameters, annual settlement at the locations of all 24 settlement markers was performed and compared to the settlement measured in 2011. Interpolation between the soil columns at the exact location of the settlement marker was performed for validation. Before Bayesian updating, our calculations tend to overestimate settlement in both 2011 and 2019 as shown in Figure B-12 and B-13. Following Bayesian updating (as shown on Figure B-20) the analysis shows an under prediction of settlement. This is to be expected because the effects of adjacent construction are not considered in the FLAC3D analysis. Section 5.3.7 should say over predict.	8/26/2019: Resolved
186	8/5/2019	MS	23, App B		Appendix B, Page 15, Section 5.4: Please clarify if the random variables are assumed to be perfectly correlated spatially (i.e., varied the same amount in each 1D soils column for each of the 6500 settlement scenarios), and discuss if this is a reasonable assumption for the 8 random variables recognized in the analysis, and if this could potentially skew the results.	JE	2019_08_07	The random variables are assumed to be perfectly correlated laterally; i.e., varied the same amount in each one-dimensional column for each of the settlement scenarios. This is consistent with the development of statistical properties of the mean soil parameters from laboratory tests on soil from various locations around the Tower, implicitly assuming the OBC to be laterally homogeneous. Also, the Tower is relatively small from a geologic perspective, so the stress and depositional history of the OBC is not likely to vary significantly across the site.	8/26/2019: Resolved
187	8/5/2019	GD	20		On S205, please confirm whether the estimated pile displacements are absolute values or relative to the top of the mat extension. If they are absolute values, shouldn't they be reported as negative values (since the jacking will push the tops of the piles downward)?	SKH	2019_08_07	The top of pile displacements are indeed intended to indicate downward movement. We have revised the table to indicate negative numbers for these displacements.	8/26/2019: Resolved
188	8/5/2019	All	26		Please confirm and briefly summarize how the concerns raised in the public service letter by L. Karp and J. Kardon (dated 7/10/2019) have been addressed in the proposed design, including: a) Effect of dewatering during construction and loss of soil due to pile construction on settlement b) Torsion caused by plan Irregularity of the new piles c) Structural integrity of existing precast piles due to mat settlement, rotation and dishing d) Use of tiebacks in the proposed construction on adjacent properties e) Capacity of existing mat foundation to resist the redistribution of gravity and seismic loads f) Significance of cracking that has been recorded the existing basement construction	ROH	2019_08_07	Supplement 188, attached hereto contains our formal letter to the EDRT chair responding to technical issues raised in Dr Pyke's communication. All of the technical points raised both by Dr. Pyke and Mr. Karp have already been addressed in an exhaustive manner as part of our original design evaluations and also in direct response to prior questions from the EDRT. Please note that we have specifically annotated in this log where specific comments relate to Mr. Karp's concerns. We restate below: a) No dewatering will occur during construction. The water table is presently only modestly above the proposed depth of excavation. We are confident infiltration of water into the excavation can be controlled by the use of jet grout improvement of the surrounding soils, as shown on the Shoring Permit Set. b) Torsion of the Tower's foundation with the perimeter pile upgrade in place has been explicitly modeled in our analyses. Although installation of the new perimeter piles results in minor shifting of the center of foundation rigidity relative to center of mass, this does not result in creation of a structural irregularity and does not produce noticeable torsional response. In fact, our analyses show that at the extremities of the foundation (the corners and sides) peak displacement in response to MCE shaking with the perimeter piles is substantially less than without them. c) Rotation of the existing mat, under the influence of tilting has indeed imposed some moment on the pile tops. This has the effect of preloading the piles with lateral demands (shears and moments). We explicitly modeled this effect in our analyses of the foundations and structure's response to earthquake shaking, considering tilting and settlements that is both double and triple that which has occurred to date and also considering bounding assumptions as to soil stiffness, and multiple suites of ground motions. With the perimeter pile upgrade in place, none of the ground motions produce sufficient additional displacement into the piles to cause failure.	8/26/2019: Resolved
								d) No new tiebacks are installed as part of the Perimeter Pile Upgrade. However, in order to perform the construction, it is necessary to cut tie backs that were installed as part of the excavation shoring for the original Millennium Tower basement and foundation construction. These tie backs ceased to serve any function once construction of the Millennium Tower basement was completed (14 years ago). They were originally intended to be temporary and sacrificial. Cutting these tiebacks will have no impact on adjacent construction or te Millennium Tower. e) We have explicitly modeled the existing piles and mat foundation together with the new piles and mat extension both in CSI Perform and CSI SAFE, industry standard tools for evaluation and design of such systems. We have evaluated the adequacy of the foundations for all load combinations specified in the building code, as well as additional load conditions evaluating what-if scenarios associated with additional building settlement and realistic modeling of earthquake effects. Our calculations demonstrate the foundation is adequate to redistribute the building weight induced by settlement and tilting as well as that associated with jacking, which tends to counter the effects of tilting. f) Most cracking reported in the "Millennium Tower" basement actually occurs in the basement walls of the adjacent podium structure and garage. It is a result of the settlement that has occurred across the site. Cracking within the basement of the Millennium Tower itself if limited and has been mapped and studied both by Arup and ourselves. The cracking in the occurs because the combined system of the basement walls, foundation mat and first floor slab act as a deep (story-high) orthotropic grillage. As the building settled and the mat foundation dished, this orthotropic grillage followed the mat's curvature and experienced stresses The mat and first floor slab experienced tension and compression stress associated with flexure of the system. The walls, acting as webs in this grillage experienced shear stress. In some locations this shear stress was sufficient to yield reinforcing steel and crack the concrete which actions relieved the accumulation of additional stress. It is worth noting that: 1- Neither the first floor slab nor mat foundation exhibit any signs of distress (cracking) 2- The core walls which provide the primary earthquake resistance of the structure do not exhibit any indications of cracking or distress 3- The core walls and mat foundation were originally designed to resist required earthquake stress without reliance on the basement walls. We conclude the observed cracking has no significant impact on the building's structural safety.	

Comments by EDRT						Responses by Design Team			Date
189	8/5/2019	CS	22		In the Specification Section 3.08C (under 3.08 Placement of Concrete) why there is a section allowing concrete to free fall down the shaft?. There will likely be groundwater infiltrating into the rock socket quickly so 3.08B, which calls for placing with tremie pipe, will probably govern. Since there are centralizers along the length of the central reinforcement, allowing concrete to free fall doesn't make sense because it will hit the centralizers and segregate as it is falling.	SKH	2019_08_05	We have revised Specification Section 31 63 29 to no longer allow free-fall of concrete during placement. Updated wording: "Concrete shall not be permitted to free fall during placement. Concrete shall placed through the use of adjustable length pipes or tubes and not allowed to strike reinforcement or other objects in the hole."	8/26/2019: Resolved
190	8/22/2019	GD/MS	20		Based on the EDRT consultation with DBI, the monitoring period continue for 10 years after completion of the construction. So, to address we would suggest changing note 5.b.v on S207 to read: "Annually for the next 8 years" (i.e., 8 years beyond the initial 2 years).	ROH	2019_08_23	We have revised S207 as requested.	8/26/2019: Resolved
191	8/22/2019	GD/MS	21		Clarify that the indicator pile testing report will also include the data analysis and synthesis and drilling criteria for production piles (so they can be reviewed by DBI/EDRT). To address this, we would suggest that the following item 8f be added to the Pile Testing Notes on Sheet T002: Add item 8f: Rock socket drill depth criteria, and anticipated rock socket drill depths of production piles.	ROH	2019_08_23	We have revised T002 as requested.	8/26/2019: Resolved
	END OF COMMENTS								