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

Review Panel Members
GD Greg Deierlein, Chair
ML Marko Schotanus, R&C
CS Craig Shields
SV Shah Vahdani

Construction Documents Phase
Issue and Revision Dates

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<thead>
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<th>No.</th>
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<td>CACIS V1 - Design Overview</td>
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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.
The pile design was updated after finalizing the current FLAC3D analysis. As the pile elements in the FLAC3D model were specified as elastic as described in the drawings (e.g., reference to 850 kip pile yield force, 18-in. dia. steel casing, 3" dia. central bar). Please refer to the latest project on file for the updated pile design.

Upon further consideration of the iterative approach mentioned above, we would like to encourage a 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 loading. First, FLAC3D is used to compute the stresses beneath the Tower due to gravity loading, including the weight of the building. For the current load, the induced stresses associated with the load carried by the new Permanent Pier. The resulting stresses are then transferred 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 further analysis of more scenarios (e.g., variability of soil properties). An iterative analysis is not possible in this manner, as such an analysis would require that the step at the beginning of the analysis be the same as the final step. In this analysis, the "input" for the start of the analysis (i.e., FLAC3D) is the building load and the "output" at the end of the analysis (i.e., from the spreadsheet) is the estimated settlement.

The difference in pressure between northwest and southeast corner is 134 psi which is less than 10% of average load of 12,000 psi. Please include in settlement calculation or provide further justification for ignoring the tilt in settlement calculations. We note that settlement this difference is due to the rotation that has occurred to the present date, which will tend to relieve some of the rebuilding that has occurred due to settlement. In a north-south aligned section through the area of most severe settlement, only very minor mohs distortion is predicted by the analyses which appear to have a better predictability of the building. The building currently tilts approximately 120 psi on the underlying soils, neglecting spread of load through the corona 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 1/2 of 16 inches (1/2 of 100 psi = 1/2 of 12 ksi) each to the east and west directions. Assuming that the building and foundation behave as a rigid body, this eccentricity imposes about 720 psi additional stress at the northwest corner, due to P-δ effects, resulting in a 3% change in pressure at the average uniform pressure without tilt. This is offset by differences in loading due to adjacent site conditions.

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Resolved
Priority Comments

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Resolutions by AF

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ROH/SKH 2019_06_07 We will include the update/revised geotechnical report, specific recommendations regarding the rock socket design basis, provisions for monitoring and geotechnical follow-up on results of the pile load test, procedures and criteria for selecting the rock socket depth and length characteristic based on observations of conditions encountered during installation and loading of the test pile, and procedures for review and approval by AOEs of significant changes to the original design basis.

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24 1/14/2019 all 5, 7 high

Please provide the information on risk class. Following up the second part of our concern, please confirm whether the feasibility of the friction coefficient has been evaluated in the evaluation of creep in the clay layers for settlement calculations.

ROH/SKH 2019_06_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.
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**Institution**: VL - Leader [LG], Section T.4. Has the MF adjustment to mat shear reinforcement (Fig. 9, pg. 199) been correctly implemented in the evaluation of the mat shear capacity? Perhaps it should be neglected in the trm (OJ, pg. 198), otherwise more analysis is needed to demonstrate that the headed bars will participate in a stress-releasing mechanism for shear. Related to this—confirm if the representation of existing headed bars is correct in S401?  
**Response**:  
We evaluated the effectiveness of shear reinforcement in the mat foundation. We confirmed that pile dowel reinforcement extends sufficiently to the location of the headed bar reinforcement in the mat and transfers to 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. 2".  
**Comment**:  
The representation of existing headed bars will be updated on S401.

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**Notes:**

1. **Comments by EDRT Responses by Design Team Date**
2. **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 the location of the headed bar reinforcement in the mat and transfers to 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. 2".
3. **5/15/2019**: We checked the shear capacity of the mat in smaller segments in our stress-strain analysis. Please refer to this May 2019 edition to the design calculation submit for results.
4. **1/14/2019**: All comments have not been addressed yet. Reiterate the need to check the influence of the shoring walls (for both the original mat construction and the retrofit) has been considered in the response to lateral earthquake effects, including possible effects on pile deformations and/or creating an eccentricity in foundation response.
5. **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 SCL and SCL-2 Table 1.
6. **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 monitoring during and after construction to include piezometers, inclinometers, pile load cells, mat vertical displacements and survey points located on the building facade.
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.

1/14/2019

Please confirm how the incline base shear forces in the podium building compare to the design base shear for the podium building, having any checks been made to confirm the safety (integrity of the shear wall) in the podium building under the induced forces.

3/11/2019

We compared the design base shear for the podium building to the demands from our analysis. The peak shear from our analysis with 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?

5/15/2019

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.

5/15/2019

We computed the podium base shear with and without the retrofit piles. They are essentially the same. Refer to Supplement No. 33 for plots of base shear demand vs. time obtained from our current analyses.

1/30/2019

We have a few comments regarding the proposed monitoring program as outlined in the new drawings (EDRT and SDH).

1/30/2019

(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 the measured deflections deviate from the analysis predictions (within a reasonable tolerance). 

1/14/2019

Please provide an estimate of the expected time (project WPA) for the soil rebound and secondary loading, as required by comment #9 on S204.

5/15/2019

We have no data available on the site where the expected rebound and secondary loading occur. Based on the literature, the rebound is expected to be approximately 1 meter, which is less than 1 inch of rebound, as noted in the comment. The rebound is expected to occur over a period of about 2 years. We will monitor the rebound and report on the rebound as we observe it.

1/22/2019

We refer to the previous sections of the Structural Engineering report on soil behavior. The comment is for jacking operations to be observed by the Structural Engineer.

1/30/2019

(1) The project geotechnical engineer estimates that approximately 1 inch of rebound will occur over a period of about 2 years. Next, to that 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 the rebound through load cell data and, make judgements as to whether jacking is necessary.

1/22/2019

The intent is to require 24 hours between each of the 100 kip load increments.

3/1/2019

We refer to the previous statement of Structural Engineering on soil behavior. The comment is for jacking operations to be observed by the Structural Engineer.

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The project geotechnical engineer estimates that approximately 1 inch of rebound will occur over a period of about 2 years. Next, to that 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 the rebound through load cell data and, make judgements as to whether jacking is necessary.
41 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

43 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

44 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

45 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

46 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

47 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

48 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

49 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |

50 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 load reduction during the transition from the at‐rest state to the train box state. This is repeated separately for each wall in question. The load reduction during the transition from the at‐rest state to the train box state is

| 5/15/2019 | Resolved. |
49 1/30/2019 SV 2 Section 1.1.2 Water is 10 feet bgs and not 3 feet. Please confirm/clarify.

50 1/30/2019 SV 2 Section 1.1.2 Water is 10 feet bgs and not 3 feet. Please confirm/clarify.

54 1/30/2019 SV 2 Section 1.1.2 Water is 10 feet bgs and not 3 feet. Please confirm/clarify.

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57 1/30/2019 SV 2 Section 1.1.2 Water is 10 feet bgs and not 3 feet. Please confirm/clarify.

58 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 tough filler metal (related to notes on Drwg. T002). 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.

59 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 tough filler metal (related to notes on Drwg. T002). 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.

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64 1/30/2019 60 7, 4 High Drwg. S406 (details 2 and 3), S402 and Cal V4, 3.1 – Please confirm if the vertical headed bar reinforcement in the "high strength" and "loggia" 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 storey tie mechanism to show how the tension force in the threaded anchor rods is transmitted back to the mat.

RND/IF 2018_02_26 The vertical headed bar reinforcement is designed to resist the maximum of the SMA strip tie demands and the SSD tie plate design load increased by a factor of 1.5. We updated the calculations in Volume 4, Section 3.5 to show both the regular and high strength detail checking starting on page 30.

65 1/30/2019 60 7 High Drwg. S406 and S502 – Please update the drawing to show the exact configuration of the vertical headed reinforcement in the southeast corner of the mat, with the bar heads above the lower layer of mat reinforcement.

RND/IF 2018_02_26 The sections have been updated. Please refer to revised drawings.

66 1/30/2019 60 7 High Drwg. S406 – Please confirm if the "Z" embedment length is sufficient to develop the PM bars.

RND/IF 2018_02_26 We reviewed our shear friction calculations. Shear friction strength is now based on the equations proposed by Mitchell et al which is referenced in the commentary on ACI 318-14 Section 22.9.1.2. For the majority of the mat extension, no spalled reinforcement is required. At the west rim extension, approximately between Gridlines D and E, additional #6 bars spaced at 16 in horizontally and 24 in vertically are required to meet the minimum reinforcement ratio correlated to Transcat's test data. The revised design requires 3.5-inch embedment length for these bars. We will update Drawings S201, S406, and S402 with the revised reinforcement. See attachment "Supplement No. 62" for design calculations.
Table 3-1: Please include designations for moment frame columns and beams in the table. In addition, for the foundation piles, please specify how much 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 girders and the new composite girders.

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 much 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 girders and the new composite girders.

Comments by EDRT Responses by Design Team Date
68 1/30/2019 GD 2 High Calcs, Table 3-2 and Calcs Table 3-1 and 3-4. The assumed expected strengths of the 10,000 psi concrete (3,000 psi unreinforced and 18,000 psi confined) are quite high. FIBER TED V2 captions that the expected strength of 18,000 psi for Day 1 is too small for high strength concrete, and AASHTO TCI currently proposes to change the factor from 1.2 for 7 days to 1.0 based on a study by Noakes and Sowersen (Calibration of design code for buildings Part I, 2000). Two additional questions are:
- Whether Member's confined concrete strength parameters apply to high strength concrete, and whether there should be a reduction factor to relate concrete cylinder strength to effective strengths in concrete member sections (Factor 6 of Member 2015 Handbook). Please confirm the basis for the assumed values, which are based on the analysis results. Members feedback will be sensitive to these assumed values.

Comments by EDRT Responses by Design Team Date
69 1/30/2019 GD 1, 3 Calcs V1, Table 3-2 and Calcs Table 3-2: The assumed expected strength of the 10,000 psi concrete is 18,000 psi (not 3,000 psi unreinforced and 18,000 psi confined). This is consistent with the as-built vertical shear reinforcement in the mat, which is anchored above the lower plate of the reinforcing cage. In addition, in Figure 3-6, please confirm that the reported values are expected strengths as opposed to nominal or design strengths for the 5 inch width of the girders beams.

Comments by EDRT Responses by Design Team Date
70 1/30/2019 GD 2 High Calcs V1, Figure 3-26 and pg. 8 (related to comment 3). Please confirm that the calculated net stress values are consistent with the in-situ vertical stress reinforcement in the mat, which is increased above the lower plate of the reinforcing cage. In addition, in Figure 3-6, please confirm that the reported values are expected values (as opposed to nominal or design strengths for the 5 inch width of the girders beams).

Comments by EDRT Responses by Design Team Date
71 1/30/2019 GD 2 High Calcs, Figure 1-26 and 3-31 and 3-35. Please confirm the definitiveness of the title, “About 5” and “About 9” in Table 29. Does it refer to gillage beams running in the N and E direction? 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 confirm that the report caption in Figure 3-26, 25 correspond to expected member strengths (versus nominal or design stress) and (2) the amount of reinforcing steel used in the calculations on pg. 39.

Comments by EDRT Responses by Design Team Date
72 1/30/2019 GD 2 High More please provide the figure it would be useful if we could add on with the in-service set of calculations. More importantly, please confirm whether the ENGO pile confines and capacities are being used as the basis for the final analysis. If not, the EDT other needs access to the ENGO values or to supporting calculations by ENGO.

Comments by EDRT Responses by Design Team Date
73 1/30/2019 GD 2 High Calcs V2, Figure 1-26 and pg. 39 (related to comment 4). Please confirm the caption in Figure 1-26, 39 shows a numerical value of “5” and others (in green) shows values of 0.5 to 0.2. Are these values of expected stresses instead of nominal stresses? If so, the expected stresses should be related to the mat load bearing capacity and effective stresses in “Supplement No. 25”.

Comments by EDRT Responses by Design Team Date
74 1/30/2019 GD 2 High Calcs V2, pg. 39. Please confirm the sentence “The designers were not aware of the ENGO analysis.” The value has been calculated.

Comments by EDRT Responses by Design Team Date
75 1/30/2019 GD 2 High Calcs V2, Figure 3-1, 3-2, 3-3, 3-4, 3-5, 3-6 and pg. 39. Please confirm how the capacities are calculated. Is this a standard building code check, where design strengths are used? Or do you use the expected strengths? Please clarify why the reported DCR values are less than 0.5. In terms, the pile shows some orange and red locations, which according to the legend are >1.0. Please provide a short written narrative to summarize and explain the implications of the mat DCR values, including: (i) sensitivity of results the foundation spring models (ENGO versus AASHTO), and (ii) effect of the foundation resistors to isolated building code requirements and performance in general.

Comments by EDRT Responses by Design Team Date
76 1/30/2019 GD 2 High Calcs V2, Figure 3-26 and 3-31: Please confirm how the capacities are calculated. Is this a standard building code check, where design strengths are used? Or do you use the expected strengths? Please clarify why the reported DCR values are less than 0.5. In terms, the pile shows some orange and red locations, which according to the legend are >1.0. Please provide a short written narrative to summarize and explain the implications of the mat DCR values, including: (i) sensitivity of results the foundation spring models (ENGO versus AASHTO), and (ii) effect of the foundation resistors to isolated building code requirements and performance in general.

Comments by EDRT Responses by Design Team Date
77 1/30/2019 GD 2 High Calcs V2, Figure 1-26 and pg. 39. Please confirm the sentence “The designers were not aware of the ENGO analysis.” The value has been calculated.

Comments by EDRT Responses by Design Team Date
78 1/30/2019 GD 2 High Calcs V2, Figure 3-1, 3-2, 3-3, 3-4, 3-5, 3-6 and pg. 39. Please confirm how the capacities are calculated. Is this a standard building code check, where design strengths are used? Or do you use the expected strengths? Please clarify why the reported DCR values are less than 0.5. In terms, the pile shows some orange and red locations, which according to the legend are >1.0. Please provide a short written narrative to summarize and explain the implications of the mat DCR values, including: (i) sensitivity of results the foundation spring models (ENGO versus AASHTO), and (ii) effect of the foundation resistors to isolated building code requirements and performance in general.

Comments by EDRT Responses by Design Team Date
79 1/30/2019 GD 2 High Calcs V2, pg. 8 (related to comment 4). Please confirm the caption in Figure 1-26, 39 shows a numerical value of “5” and others (in green) shows values of 0.5 to 0.2. Are these values of expected stresses instead of nominal stresses? If so, the expected stresses should be related to the mat load bearing capacity and effective stresses in “Supplement No. 25”.

Comments by EDRT Responses by Design Team Date
80 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.

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 is expected over the next 1½ years.

82 1/30/2019 CS 5 Section 8.0 References ENGEO (2018) report/memo ... The report will be revised to be self-sufficient. All references to ENGEO will be removed.

83 1/30/2019 CS 5 Section 9.2 How was settlement from secondary compression analyzed using 1D analysis? JE ... 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 compression settlement calculation using a = 1.0. We estimated k by a through geotechnical theory and observed the settlement on the site. The height of each FLAC3D zone...

84 1/30/2019 CS 5 Section 9.2 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

85 1/30/2019 CS 5 Section 9.3 What is a “displacing” or “deflecting”? Is this the jet-grout columns below the excavation? Maximum lateral deflection for the proposed design should be less than 100 psi, and should be less than 0.25 psi.

86 1/30/2019 CS 5 Section 9.3 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
1. The attached calculation "Supplement 116" includes breakdown of the loads and design strengths of each strip. 

2. 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.

3. 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 becoming a part of the excavation.

4. 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. 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 two O-cells, 3) mid-depth between two O-cells, 4) mid-depth between O-cells and the soil, 5) mid-depth between O-cells and the soil, 6) pile tip. We do not plan on incorporating an additional O-cell.

6. ROH/LH 2019_03_06 Refer to Supplement No. 112 Appendix A for the Technical Paper by Canbolat et al.

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. ROH/LH 2019_03_06 See the figures in Supplement 115 for the pile loading.

116 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. ROH/LH 2019_03_06 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 SAFE?

117 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. ROH/LH 2019_03_06 The analysis using 10 ft strips was performed to understand the stress distribution in the mat foundation before and after the retrofit. 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 load and foundation spring bounding analyses.

118 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. ROH/LH 2019_03_06 2. The attached calculation "Supplement 116" includes a comparison of the factored shear and flexural demands and design strengths of each strip. The DCs on pg 7 of the attached Supplement 116 are for the north strips. We corrected the labels in these summary tables. We also 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.

119 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. ROH/LH 2019_03_06 The analysis using 10 ft strips was performed to understand the stress distribution in the mat foundation before and after the retrofit. The strip width is one foot equal to the mat depth and is not likely that failure of the mats 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.
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, the sloped new 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 edge shear. We assumed a sudden drop to zero hinge strength 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 failure or column collapse. Please reffer to the attached "Supplement No. 120" for additional discussion.

We compare their moment-curvature results to ours in the attached "Supplement No. 118". Based on the comparison, accounting for the initial strength would not negatively affect pile behavior. These demands are likely to increase by about 50% but would remain less than the shear strength of the piles.

We previously discussed the analysis using 10D piles to better understand the stress distribution in the mat foundation before and after the retrofit. The adequacy of the mat was demonstrated in calculations shown in Volume 5, 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 a foot of the edge of the mat as described above. The existing mat still maintains shear forces away from the columns 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 not much exceeding 1.0 for most strips. In previous discussions, finite element models that assume linear elastic behavior?

We assume 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 failure or column collapse. Please refer to the updated Calculation Volume 3, Section 5.1 – revision 5.

126 1/30/2019 SV CSA March 1, 2019 memo presents data regarding differential settlement contours between 2009 and 2018. Data indicates substantial rigid body rotation toward front street has occurred during the time period. The data presented on Figure 14 concerning pile head rotation assumptions used in dynamic response analysis? Please confirm as discussed in meeting on 4.23.2019.

127 5/15/2019 SV The monitoring Program reported below was presented in a special meeting 4.23.2010. Please confirm if the current system and the program has been incorporated in the latest design drawings and specifications.

- 25 instances of monitoring of the basement and exterior perimeter and extremities for two years. This includes travel time and costs, time on site and pilot and is broken down as follows:
  - 6 months of bi-weekly readings (320)
  - 1.5 months between monitoring (8) and Quarterly monitoring for 15 months (5).
- 76 instances of prism and basement monitoring review and analysis based on the following schedule:
  - 3 months of weekly monitoring immediately prior to, during, and following construction (32);
  - 8 months of bi-weekly monitoring (96);
  - 1.5 months between monitoring for two years (36); Quarterly monitoring for two years (8); and
  - Annual monitoring for five years (5).

Please confirm as discussed in meeting on 4.23.2019.
Referring to proposed monitoring program (Comment 17), please (a) clarify how and in what ways plans 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 preserving the period of weekly monitoring as an absolute number of readings (50), 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 5/30/2019.

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Please confirm if the production volume is measured at the top of the mat or at the base of the mat. The fact that the mat can sustain any yielding that occurs as a result of nonuniform propagation of vertical bedding would be up to the mat.

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Please provide MCE peak vertical displacement at the top of the mat by converting the peak horizontal displacement obtained at the site response analysis (SSM) data to horizontal using the North-Armstrong & Guadalupe (2012) v/h relationship. This results in a peak estimated displacement in the top of risk of 12.0 inches. Assuming that the mat does not move, this results in a strain of approximately (12.0/50)=0.024 over the free length of the mat. We note that the peak horizontal displacement was converted from the top of the mat along Mission and extreme south end of the mat along Fremont. This result is in addition to 0.1 parts per million strain of approximately 0.075. We note that the mat material has been successfully tested statically to a strain of 0.2, which is substantially in excess of the projected total strain. Regardless, we acknowledge, the graph (though not fully reversed cyclic) remains relative to the seismic loading, that there is a small possibility of failure of some rocks, given MCE shaking. This would not result in building damage, but would require repair.

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Please clarify if the specification at 3.03C1 addresses the following: (1) strength for wells/boreholes/strainers in the specification (500-750 psi) does not match Sheet T002 (500 psi max); (2) complete sentence on 3.04C1; (3) Section 3.04F2 please review the annotation on 3.04C2 as this clarifies the condition of the rock socket sides and bottom will be observed once drilled shaft will be filled with drilling fluid.

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2019_08_05 The monitoring plan on S207 indicates that monitoring will continue for 5 years, but DBI Information Sheet IS-18 (2019) notes that the monitoring will end. Please refer to the updated sheets on monitoring.

**Please refer to the updated sheet S402.**

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2019_07_19 We refer to the updated sheet S502. **Please refer to the updated sheet S503.**

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We do not see any provisions made for filling the annulus between the outer and inner casing of the upper region of the shaft after jacking up. Please specify sample and procedure for grouting, including provisions to check (and if necessary, rework) the formation of voids in the grout.

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We did not see any definitions for terms like “slurry” or “drilling method” in this comment, which is not a major issue as we only are referring to the vertical bore. However, we would suggest adding to the revised drawings for the “drilling method”.

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We are not intending to use the mat slab as a single shaft and high strength concrete is not to be used in the slabs. We refer to the updated sheet S504.

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On S401, please confirm if the use of Sonotubes in the mat extension of the SLT is intended to fully develop the bars, or if it is intended to use the bars for nonstructural purposes. If the intention is to use the bars for nonstructural purposes, then the bar shall be removed with a hammer and chisel.

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Please refer to the current final project report.

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We refer to the updated sheet S505. **Please refer to the updated sheet S506.**

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We refer to the updated sheet S506. **Please refer to the updated sheet S504.**

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We refer to the updated sheet S504. **Please refer to the updated sheet S505.**

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We refer to the updated sheet S505. **Please refer to the updated sheet S504.**

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We refer to the updated sheet S506. **Please refer to the updated sheet S505.**

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We refer to the updated sheet S505. **Please refer to the updated sheet S504.**
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 Tower footprint. The Tower footprint will remain normally consolidated, but the state of stress will be significantly reduced, to the extent that future settlement from 2020 to 2060 is expected to be less than an inch. 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 consistent with the assumed rock strengths to obtain the desired outcome of the tests.

We estimate a resistance on the order of an additional 8 inches would ever occur at the locations of these new piles. It is extremely unlikely that settlement on the order of an additional 8 inches would ever occur at the locations of these new piles.

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 consistent with the assumed rock strengths to obtain the desired outcome of the tests.

Since the friction is assumed to act against the volume of jet grout Column. Please address the effects of heave on adjacent streets.

We are using a guide water stop to maintain the nominal column volume 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 2060 and 2020, respectively. Please clarify whether the eccentricity of frictional resistance is considered in the structural analysis model, but please clarify whether the eccentricity of the resistance is considered.

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Since the friction is assumed to act against the volume of jet grout Column. Please address the effects of heave on adjacent streets.

We are using a guide water stop to maintain the nominal column volume 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 2060 and 2020, respectively. Please clarify whether the eccentricity of frictional resistance is considered in the structural analysis model, but please clarify whether the eccentricity of the resistance is considered.

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 Tower footprint. The Tower footprint will remain normally consolidated, but the state of stress will be significantly reduced, to the extent that future settlement from 2020 to 2060 is expected to be less than an inch. 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 consistent with the assumed rock strengths to obtain the desired outcome of the tests.

Since the friction is assumed to act against the volume of jet grout Column. Please address the effects of heave on adjacent streets.

We are using a guide water stop to maintain the nominal column volume 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 2060 and 2020, respectively. Please clarify whether the eccentricity of frictional resistance is considered in the structural analysis model, but please clarify whether the eccentricity of the resistance is considered.
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### Appendix B, Page 6

#### Section 6.1

As noted, the geotechnical settlement calculations assume a constant 800 kip/pile jacking force. This interaction between jacking, rebound, and underlying strata is complex. The pilehead settlement results in stress on soil columns that are adjacent to the pilehead, which increases the jacking force in the pile. This process would repeat until an equilibrium is achieved.

#### Section 6.2

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 measured quality of fit over the entire mat surface based on fit to all 24 settlement markers. The validation and Bayesian updating is based on all 24 settlement markers. The results presented measured quality of fit over the entire mat surface based on fit to all 24 settlement markers.

#### Section 6.3

The random variables are assumed to be perfectly correlated spatially i.e., varied the same amount in each one-dimensional column for each of the GSSD settlement scenarios, and discard if this is a reasonable assumption for the random variables recognized in the analysis, and if the possible combinations of the variables are:

- a) Effect of dewatering during construction and loss of soil due to pile construction on settlement
- b) Torsion caused by pile rigidity of the new piles
- c) Structural integrity of existing perimeter piles due to mat settlement, rotation and twisting
- d) Use of methods in the proposed design on adjacent property
- e) Capacity of existing mat foundation to resist the redistribution of gravity and seismic loads
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### Appendix B, Page 8

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#### Section 5.1

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### Appendix B, Page 10

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### Appendix B, Page 12

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### Appendix B, Page 14

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#### Section 5.8

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### Appendix B, Page 16

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### Appendix B, Page 18

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### Appendix B, Page 20

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### Appendix B, Page 26

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<table>
<thead>
<tr>
<th>No.</th>
<th>Date</th>
<th>Reviewer</th>
<th>Priority</th>
<th>Comment</th>
<th>Response by Design Team</th>
<th>Resolved Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>189</td>
<td>8/5/2019</td>
<td>CS</td>
<td>22</td>
<td>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.</td>
<td>SKH 2019_08_05 We have revised Specification Section 3.08C to no longer allow free fall of concrete during placement. Updated wording: &quot;Concrete shall not be permitted to free fall during placement. Concrete shall be placed through the use of adjustable length pipes or tubes and not allowed to strike reinforcement or other objects in the hole.&quot;</td>
<td>8/26/2019 Resolved</td>
</tr>
<tr>
<td>190</td>
<td>8/22/2019</td>
<td>GD/MS</td>
<td>20</td>
<td>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: &quot;Annually for the next 8 years&quot; (i.e., 8 years beyond the initial 2 years).</td>
<td>ROH 2019_08_23 We have revised S207 as requested.</td>
<td>8/26/2019 Resolved</td>
</tr>
<tr>
<td>191</td>
<td>8/22/2019</td>
<td>GD/MS</td>
<td>21</td>
<td>For the indicator pile testing report will also include the data analysis and uncertainty 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.</td>
<td>ROH 2019_08_23 We have revised T002 as requested.</td>
<td>8/26/2019 Resolved</td>
</tr>
</tbody>
</table>

END OF COMMENTS