October 13, 2023

Jacob Halverson
City of Mercer Island
9611 SE 36th St
Mercer Island, WA 98040

Subject: **Hong/Kao Residence Remodel/Addition and new Detached Accessory Dwelling Unit**
5425 W Mercer Way
Permit No. 2306-124
Quantum Project #23127.01

Dear Jacob Halverson:

We have received the correction notice on the above-referenced project dated 9/23/2023. Below are our responses to the structural comments. All changes to the drawings due to a plan check comment have been clouded on the drawings.

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STRUCTURAL

General:

1. The framing plans on Sheets S.2 – S.4 are stamped by the engineer of record, but the Foundation Plan, Sheet S.1, is not. Please stamp the Foundation Plan since it contains important structural information.

   Please see the revised sheet S.1 for the requested stamp.

2. The Mercer Island Cover Sheet notes that special inspection of concrete is intended. Is that true? The Structural Notes do not indicate this.

   Please see the updated sheet S1.0, Note 15, where concrete has been added to the list.

3. Sheet S1.0 specifies Grade 40 reinforcing at #4 bars but does not address the grade of #5 bars. Please specify. For example, refer to the #5 bars specified in the structural slab of the DADU (page 224 of the calculations) or grade beams (Detail 10/S3.0 and page 225 of the calculations).

   Please see the updated sheet S1.0, Note 23, where the #5 rebar grade has been added.

4. Please clarify how shear wall capacities were determined and verify compliance with the 2018 IBC and 2015 SDPWS. For example, page 210 of the calculations identifies resistance factors used for LRFD for both seismic and wind loading. Per SDPWS 4.2.3, the LRFD factored unit resistance is to be determined by multiplying the tabulated nominal unit shear capacity, modified by applicable footnotes, by a resistance factor of 0.80. Since HF framing is used, the specific gravity

adjustment factor must be applied (0.93) per SDPWS Table 4.3A, footnote 3. It seems that wind shear capacities are too high, in general. Please verify that designs meet the capacity limitations, particularly where wind loads may govern at the DADU.

Shear wall capacities shown in the calculations follow 2018 IBC and 2015 SDPWS. Capacities are determined by multiplying the nominal wind and seismic capacities by the corresponding resistance factor. The shear demands are then divided by the aspect ratio reduction factor and specific gravity adjustment factor to get "adjusted" shear values. Since the LRFD wind capacity is 1.6x larger than the LRFD seismic capacity (0.8/0.5=1.6), to determine the controlling shear the spreadsheet compares the adjusted wind shear to 1.6x the adjusted seismic shear and selects the greater value. This shear demand is then compared with the corresponding LRFD shear capacity.

For example, SW N.1 shown on page 210 of the calculations has a wind shear load of 60 plf. This is divided by the aspect ratio reduction factor of 1.0 and the specific gravity adjustment factor of 0.93 to get an adjusted wind shear of 64.5 plf. For SW-6, the LRFD shear wall wind capacity is 696 plf so the wall is within capacity for wind loads. However, the adjusted seismic shear controls for this wall so the spreadsheet shows the seismic LRFD shear wall capacity of 435 plf.

Gravity – Main Structure:

5. IRC R703.8 establishes limitations on height of brick veneer in Seismic Design Category D2. It appears the design exceeds the prescriptive limitations which would require justification of compliance with TMS 402 – 2016 Building Code for Masonry Structures.

Masonry veneer and anchorage shown in detail 9/S4.4 comply with TMS 402, Section 12.2.2. Per Section 12.2.2.6.1: "Anchored veneer with a backing of wood framing shall not exceed 30 ft". Also, per the section commentary: "These requirements are similar to those used by industry and given in model building codes for years."

6. UB11 is supported at the north end by UB10 (page 12 of the calculations). We do not find the reaction load from UB11 considered in the design of UB10 (see page 98 of the calculations). This load would occur at the end of the 7' cantilever. Please verify design.

Please see the supplemental calculation sheet 46 for the updated calculation for beam UB10.

7. Please clarify framing at the Master Bedroom deck area. Page 12 of the calculations shows the framing that coordinates with Sheet S.3. The UJ6 joists are supported on the north at UB10, but it is unclear what supports the south end of the beam. There is an HSS member in plan, but we are unable to find the calculations. Also, the east end of this HSS member frames into an LVL that we do not find evaluated in the calculations. This LVL is supported on the south end on UB13, but that point load does not appear to have been considered in the design of UB13 per page 55 of the calculations. Please justify design of these framing members and please cut details on Sheet S.3 at the south end of the deck to clarify framing conditions.

Please see the supplemental calculation sheets 74 to 75 and 85 to 88 for the Master Bedroom deck area. Beams UB10b and UB7b have been added for a comprehensive assessment of the framing design.

8. Please verify reinforcing in the concrete plinth to confine reinforcing and anchor bolts at the top of the column in Detail 7/S3.1 per ACI 318 10.7.6.1.6. Transverse reinforcement must be distributed within 5" of the top of the column or pedestal and must consist of at least (2) #4 or (3) #3 bars.

   Please see the updated detail 7/S3.1 where an additional #3 tie has been added as requested.

9. Page 5 of the calculations identifies the dead load of the main floor as 15 psf; however, it appears that typical floors were evaluated with a dead load of 12 psf. For example, see page 41 of the calculations. Please verify design.

   Please see the supplemental calculation sheets 36 to 88. Pertinent members have been revised for correct floor dead load of 15 psf.

10. Please verify the reaction loads at Beam UB2a are adequately supported at the foundation.

    Please see the supplemental calculation sheets 8 to 10 for the spread footing 'F1' analysis at the requested location.

11. RB15 supports an exterior wall above as well as part of the existing roof above. The calculation on page 38 does not appear to consider the existing roof. Please verify adequacy.

    Please see the supplemental calculation sheet 64 for updated calculation for RB15.

12. We are not finding the calculations for the 2x12 deck joists on the Main Floor Framing Plan, Sheet S.2. Please clarify where those are evaluated.

    Note that the originally submitted Joist J1 calculations on page 36 show the design for the main floor deck joist. This page is included in the supplemental calculations for reference.

13. Please cut a detail on Sheet S.3 along Grid 4 on the Upper Floor Framing Plan to show connection details and uplift connectors at the new garage door headers.

    Please see revised drawing sheet S.3 and new detail 8/S4.2 for the requested detail.

14. There are some significant dead loads that do not appear to have been evaluated. Please review the following and provide additional information:

    a. Please provide a foundation key plan with significant dead loads identified with supporting calculations.

    Please see the supplemental calculation sheets 3 to 32 for the revised foundation design.

    b. Refer to the HSS columns supporting B7 at the main floor along Grid C that are supported on an existing footing. It does not appear that the existing footing has sufficient capacity. Please evaluate.

This location has been revised to a single HSS column supported on a larger spread footing. Please see revised sheet S.1 and the supplemental calculation sheets 17 to 19 for the spread footing 'F3' analysis.

c. We do not find calculations for the framing around the new exterior stair or the HSS column support. Please provide.

Please see supplemental calculation sheets 54 to 61 for the exterior stair framing analysis.

d. We do not find calculations that evaluate the foundation supporting the W16x67 column at the living room frame.

Please see the supplemental calculation sheets 23 to 26 for the spread footing 'F5' analysis at the requested location.

e. Please verify adequate support is provided for the north end of UB2a at the foundation.

Please see the supplemental calculation sheet 4 for the existing wall footing analysis at the requested location.

Lateral – Main Structure:

15. Please verify that the requirements of SDPWS Section 4.2.5.2 for open front structures have been met at locations where the roof diaphragm cantilevers beyond the lateral resisting elements. We are most concerned with forces in the N/S direction on the west side of the structure. An explanation of your design philosophy would be helpful.

In the N/S direction, roof and floor diaphragms have been designed as cantilever diaphragms per SDPWS Section 4.2.5.2. Additional shear walls along Grid 2 have been added to the main floor and basement levels to meet the diaphragm aspect ratios and reduce torsional forces on the structure. Please see the revised sheets S.2, S.3, and S.4 and supplemental calculation sheets 89 to 124 for the lateral analysis.

16. How are lateral forces at the decks on the west side of the structure resisting lateral loads? Please evaluate.

The decks on the west side of the structure are tied into the floor diaphragms of the house. The chord forces are resolved via straps and continuous beams as shown on the updated plans. Please see supplemental calculation sheets 123 and 124.

17. Redundancy must be evaluated per ASCE 12.3.4. There are basically two options; use ρ=1.3 or justify that ρ=1.0 is appropriate. Provide supporting calculations to justify.

Seismic forces have been increased, using ρ=1.3. Please see the supplemental calculation sheets 89 to 118.

18. Provide an evaluation of horizontal diaphragms to show adequacy per IBC 2305.1, SDPWS Section 4.2.1, and ASCE Section 12.10.1. Diaphragms must be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and re-entrant corners, the design must assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

    Please see the supplemental calculation sheets 119 to 122 for analysis of controlling diaphragm forces in the structure. Horizontal straps have been added at diaphragm discontinuities. Please see the revised sheets S.2 and S.3.

19. Per ASCE 12.10.2, collector elements must be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. For example, refer to the shear wall adjacent to the stair. Will drag members and straps be provided?

    Drag members and straps have been provided at the stair openings to transfer the lateral forces in the structure. Additionally, the diaphragm has been blocked in this area to ensure the shear walls share the loads.

20. Horizontal diaphragm shear forces must be distributed to the vertical resisting elements based on tributary area for flexible diaphragms per SDPWS 4.2.5 or otherwise justified by the engineer of record. There are several locations where shear wall segments are assumed to act along the same line, but in reality they are separated by 10' or more. For example, refer to SW 4.3 & 4.4 on the upper floor and SW 3.5 & 3.6 on the main floor.

    Horizontal diaphragm forces have been distributed to the shear walls based on tributary areas. At certain locations, shear walls have been designed to act along the same grid line based on engineering judgement and anticipated behavior of the structure under lateral design loads. Along Grid 3, the diaphragm has been blocked between shear walls to create added stiffness that will distribute the lateral loads between walls.

21. All shear forces in the basement in the N/S direction are assumed to be attributed to the east wall line. This would induce a significant amount of torsion in the diaphragm, require increase loads due to redundancy issues, pose irregularity issues, and would require additional load path details. Please see above comments and justify design. It appears at least one other line of resistance on the west side of the diaphragm will be needed.

    Additional shear walls along Grid 2 have been added to reduce torsional forces on the structure. Please see revised sheets S.2 and supplemental calculation sheets 89 to 124 for the lateral analysis.

22. We are unable to complete our lateral review due to the scoping comments above that need to be addressed. Additionally, we are not finding clear details of load path to many shear walls. We are providing a few examples, below; however, additional comments may arise once

resubmitted. Please apply the concepts in the comments below to other locations. Details should be provided at all shear walls to clarify load path.

Please see revised sheets S.2, S.3, and S.4 and supplemental calculation sheets 89 to 124 for the updated lateral design. Structural details have been added to the plans to clarify the load path.

23. Detail 3/S4.5 is cut on Sheet S.4 at SW 3.7 on Sheet S.4. This is not the correct detail. Please clarify load path for lateral forces.

Please see the revised sheet S.4 where the correct detail 11/S4.2 is now cut at the location noted.

24. Detail 9/S6.0 is cut on Sheet S.4 at SW C.7 on Sheet S.4. While this detail shows the connection of the WF beam to the upset GLB, it is not showing how shear is transferred into the shear wall below. Please clarify load path for lateral forces.

Please see the revised sheet S.4 where the new detail 9/S4.3 has been added to show the shear transfer, etc. at the location noted.

25. SW 4.1 has a significant shear force attributed to it. How are shear forces getting into this wall? It is noted on Sheet S.3 that there is a skylight above which is reflected in Detail 9/3.31. While Detail 2/S4.5 is cut in plan on Sheet S.3, that is not the actual condition. The horizontal diaphragm would also need a drag strut to transfer shears into this wall. Please evaluate.

The roof skylight was eliminated at this location. Please see the revised sheet S.3.

26. How are shear forces transferred into SW 4.2 at the front walls of the garage, Sheet S.3? Please cut a detail above this shear wall and consider drag members and straps as needed.

Please see the revised sheet S.3 where the new detail 6/S4.4 has been added to show the shear transfer, etc. at the location noted.

27. Where upper floor shear walls are designed with holdowns (for example, see SW 3.6 on Sheet S.3), a continuous load path should be provided to the foundation per SDPWS 4.3.6.4.4. Elements resisting shear wall forces contributed by multiple stories should be designed for the sum of forces contributed by each story.

Holdowns at the foundation have been added to provide an adequate load path for 'SW 3.6'. Please see the revised sheets S.2 and supplemental calculation sheets 89 to 118 for the updated shear wall design.

28. The west end of SW B.1 appears to be missing a holdown. See page 169 of the calculations.

Please see the revised sheet S.2 where an HDU5 holdown has been added to the west end of shear wall 'SW B.1'.

29. Please evaluate anchor bolt connections at existing foundations.

All shear walls at existing foundations will be anchored to the foundation with 5/8" Ø Simpson Titen HD screw anchors per 1/S4.0 and the details on sheet S3.1. These post-installed anchors have adequate capacity to substitute for conventional 5/8" Ø anchor bolts.

Gravity – DADU:

30. A roof dead load allowance for PV panels was not included in the gravity design of the DADU. Please clarify this in the Structural Notes, Sheet S1.0.

The DADU calculations do include a 5 PSF allowance for PV panels. The calculations for the DADU used a Dead Load of 18 psf instead of the 20 PSF noted in the design criteria. The DADU roof joists weigh 1 PSF less than the 2.1 PSF noted in the criteria and the roofing is also 1 PSF less. In addition, the percentage difference between the total load of the criteria vs. what is used in the DADU calculations is 50/48 = 1.04. the most heavily stressed roof member is Joist #2, which is only at 92% of capacity. Therefore, the DADU roof can support a 50 psf total load and no revisions to the Notes on Sheet S1.0 are needed.

31. Page 183 of the calculations identifies Roof Beam #6 which is evaluated on page 190 of the calculations. This is a roof beam that supports tributary loads from Roof Joist #1B and #1A, but the loads included on the calculation appears to only account for the self-weight and 1’ of tributary floor load. Please revise design.

Typically, we would not provide a calculation for a beam such as this that was selected to be compatible with the joists and hangers. As noted, the calculation incorrectly listed the tributary as 1’ of floor loading (52 PLF). The correct tributary loading is 9.5’ of roof loading (475 PLF), an increase of 475/52 = 9.1. The maximum result is therefore the member reaction, which is: 7% x 9.1 = 64% - OK. The moment is only 18%.

32. It does not appear that the roof sheathing thickness is called out. See Roof Framing notes on DADU Sheet 2.0.

Please see the revised ROOF FRAMING NOTE 2 on DADU Sheet 2.0.

33. Please proofread the legend on DADU Sheet 2.0. We are unclear where the 2x8 stud walls vs. brick veneer is intended. An accurate hatching description should be provided. Coordinate with the calculations starting on page 196.

Please see the revised legend on DADU Sheet 2.0.

34. Please verify structural slab design coordinates with page 224 of the calculations. It appears that the slab is designed assuming 4,000 psi concrete but only 2,500 psi concrete is specified in the Structural Notes, Sheet S1.0.

Please see the revised DADU Plan 2.0 where the concrete strength has been specified as well as the revised Structural Note 22 on Sheet S1.0 where additional notes have been added to clarify the concrete strength at the DADU.

35. At the open floor area, evaluate the exterior wall for combined axial and lateral loads per SDPWS 3.1.1 and ASCE 7-16 Section 2.3.1. Typical construction would create a hinge at the plate break (see Detail 7/S4.2). Either continue the studs from floor to roof and provide supporting calculations for these vertical members, or design framing to resist the applied forces.

The framing is designed for the opening. The open floor area has a horizontal clear span of 11'-2". Detail 7/S4.2 provides for a minimum of (3) 2x6 flat members. The components and cladding pressure is less than 29.1 PSF * 0.6 = 18 PSF. With a 9' tributary (conservative since the stairs and landing also provide bracing), the load is 162 PLF. The bending stress in the 2x6 framing is then 1,336 psi which is less than the allowable 1,768 psi for HF#2.

Lateral – DADU:

36. The ADU Roof Framing Plan on DADU Sheet 2.0 shows the E.1 & E.2 shear walls as short segments (SW-2); however, this wall at the roof level would be the full width of the double-high garage (approximately 16'). Please clarify. Also, the end studs should be full height studs; otherwise, holdowns between the upper and lower levels should be provided. Please coordinate with pages 215 – 218 of the calculations.

The wall panels are not shown as full width over the garage door because the garage door is 12'-0" tall (see Sheet S3.0) and the stud wall is 18' tall. Horizonal blocking and strapping are provided at the Upper Floor Framing Plan to transfer the loads around the door opening. Engineering judgment was used to model the building for load distribution. The resulting design shown on the permit drawings is conservative.

37. Pages 212 – 213 of the calculations show the shear force on the main floor level for the Grid C shear walls. The calculations evaluate three segments (C.1, C.2, & C.3 per page 205). The ADU Upper Floor Framing Plan on DADU Sheet 2.0 only shows two wall segments. While the change to SW-4 is probably fine, please reevaluate overturning and holdown size.

Please see the supplemental calculations sheets 125 to 138. The shear wall design at Grid C has been revised to exactly match the framing plans. Also note that the shear walls have been relabeled for clarity as shown on the shear wall key plan located on the supplemental calculation sheet 126.

38. Pages 212 – 213 of the calculations show the shear force on the main floor level for the Grid S.1 shear wall. The calculations assume an 8.5' shear wall length; however, the wall appears to be less than 6.5' in length. See the ADU Upper Floor Framing Plan on DADU Sheet 2.0. We assume the wall does not continue through the south exterior wall. Please reevaluate the shear wall capacity and holdown design.

Please see the supplemental calculations sheets 125 to 138 and the updated DADU Sheet 2.0. The shear wall design of the old S.1 wall (now labeled as S.2) has been revised to exactly match the framing plans. Note that the shear walls have been relabeled for clarity as shown on the shear wall key plan located on the supplemental calculation sheet 126.

39. Where upper floor shear walls are designed with holdowns, a continuous load path should be provided to the foundation per SDPWS 4.3.6.4.4. Elements resisting shear wall forces contributed by multiple stories should be designed for the sum of forces contributed by each story. For example, refer to the holdowns at the upper-level E.3 and C.1/C.2 shear walls.

    Please see the supplemental calculations sheets 125 to 138 for the revised lateral analysis and see the updated DADU Sheet 2.0. Some shear walls have been eliminated and others adjusted in order to provide holdowns as required for all shear walls in a complete load path to the foundation.

40. Page 220 of the calculations appears to require HDU2 Holdowns at each end of the lower-level W.1 shear wall. We do not find these holdowns specified on the Foundation Plan, DADU Sheet 2.0.

    The conservative uplift value shown on page 220 will be resisted by the perpendicular walls at each end of the 28.5' long shear wall. Note that the perpendicular walls do have holdowns at their ends.

41. Detail 9/S4.5 is cut along Grid C at the roof at the exterior wall. Please cut the applicable detail at the interior segment of shear wall along this grid line.

    Please see the updated DADU Plan Sheet 2.0 where detail 8/S4.5 has been cut at the requested location.

42. The Shear Wall Schedule, 8/S4.0, has a column for the top plate attachment. The A35 or LTP4 clips should be shown in all details at the roof to clearly show shear flow. Please show the connector, where referenced, in Details 9 & 10/S4.5.

    Please see the updated details 9 & 10/S4.5 where the A35 clips are now shown graphically as requested.

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Please feel free to call me at 206-957-3900 if you have any questions regarding our responses.

Sincerely,
Quantum Consulting Engineers, LLC

Scott Tinker, P.E., S.E., Principal
Project Manager